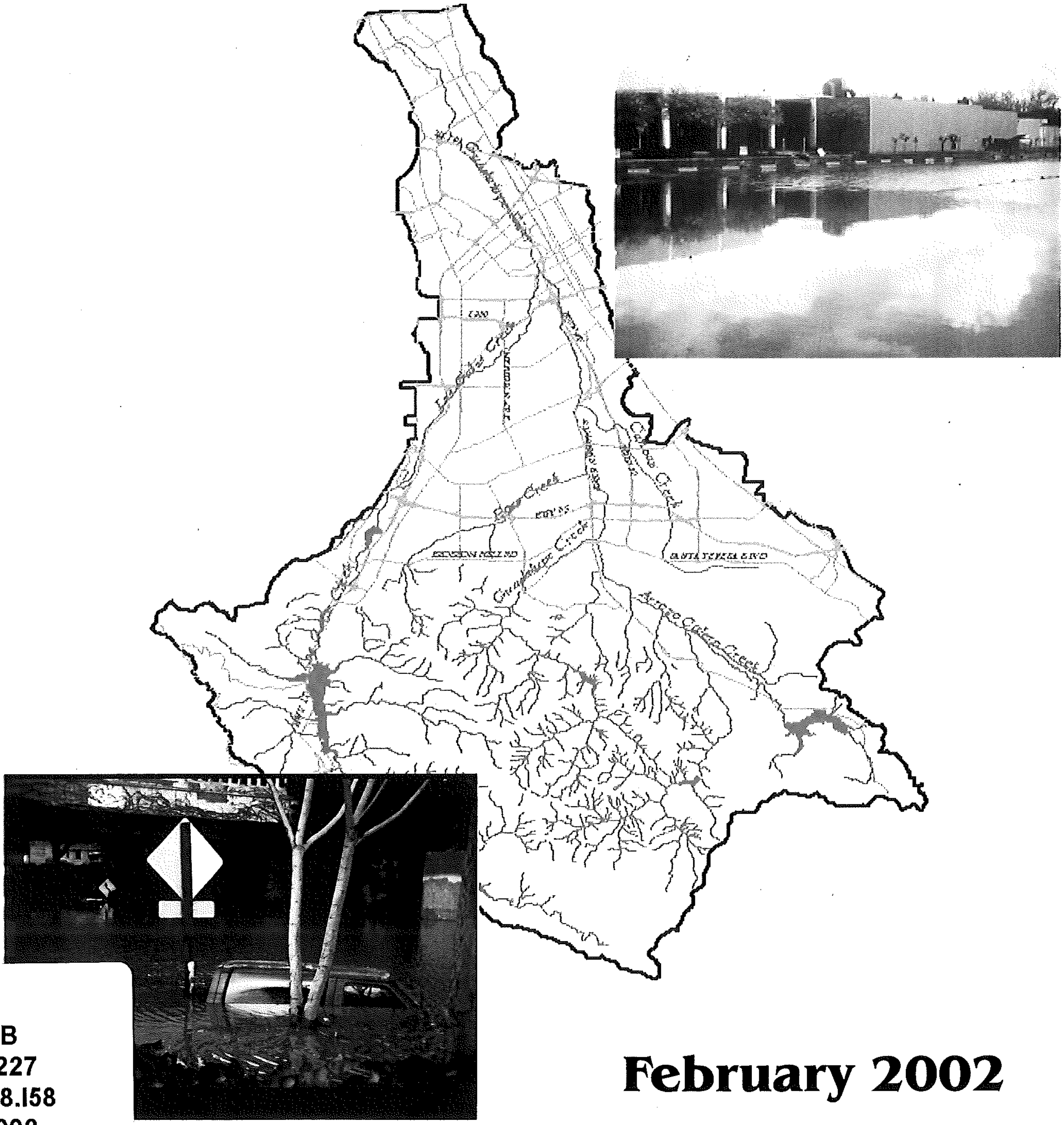
Investigation of the Long-Term Feasibility and Potential Benefits Achievable by Reduction of Peak Flood Flows within the Guadalupe River Watershed



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February 2002

Santa Clara Valley Water District

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SANTA CLARA VALLEY WATER DISTRICT

INVESTIGATION OF THE LONG-TERM FEASIBILITY AND POTENTIAL BENEFITS ACHIEVABLE BY REDUCTION OF PEAK FLOOD FLOWS WITHIN THE GUADALUPE RIVER WATERSHED

R11665

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I. PURPOSE

This study was initiated to comply with a special request from the San Francisco Bay Regional Water Quality Control Board (Regional Board) for their Water Quality Certification/Waste Discharge Requirements (WQC/WDR) issued for the Guadalupe River project and the Guadalupe Creek Restoration project (Order No. 01-036).

Requirement No. 7 of the WQC/WDR requests a special study to address:

a. In order to maximize the potential for protection of beneficial uses of Waters of the State within the Guadalupe River Watershed, additional studies are needed to determine the long-term feasibility and potential impacts achievable by reduction of or other control of peak flood flows within the watershed. Needed studies include: (1) evaluation of the peak flood flow reductions or controls needed in order, in the long term, to restore habitat function and value by retention of existing riparian habitat and/or replacement of existing concrete/rockwork with suitable riparian and shaded riverine aquatic vegetation, (2) evaluation of the potential impacts on peak flood flows of collaborative efforts with stakeholders to integrate land use, flood control, and watershed planning so as to prevent or control increases in peak flood flows resulting from new development, (3) evaluation of potential impacts on peak flood of retrofitting existing development with on-site storage/infiltration facilities for peak flows, and (4) evaluation of the potential for combinations of floodplain expansion, reservoir storage, reservoir operations, and existing and new development on-site storage/infiltration requirements to reduce or control peak flood flows and preserve/restore riparian habitat.

b. By March 31, 2002, the Santa Clara Valley Water District (District) shall submit a report acceptable to the Executive Officer summarizing the following information:

    (1) An evaluation of alternative methods of reducing or otherwise controlling peak flows in the Guadalupe River Watershed. The evaluation may be based on existing information supplemented by additional studies as needed. Peak reduction methods addressed in the report shall include on-site storage/infiltration facilities, reservoir storage, and reservoir operation modifications. The report shall address the technical, economic, and institutional feasibility, and the potential environmental impacts, of alternative peak-reduction and control measures.

    (2) An overview of land-use planning practices and guidelines currently in effect in the Guadalupe River Watershed and recommendations for modifications to those practices and guidelines consistent with the goals of riparian habitat preservation and restoration.

II. BACKGROUND OF ADDITIONAL REGIONAL BOARD CONCERNS

Subsequent to receiving Order No. 01-036, District staff discussions with staff from the Regional Board revealed that the concern behind their special study request was their belief that increased volumes and velocity of runoff from newly-developed urban areas can greatly accelerate the erosion at downstream watercourses causing declines in the biological integrity and physical habitat of streams. The concept is that stream channels tend to maintain a state of quasiequilibrium with the flow regimen of the stream such that there is a constant frequency of overbank flow. A stream's effective discharge is that discharge which transports the largest fraction of the annual sediment load over a period of years. The recurrence interval of the effective discharges in the Yampa River basin, Colorado and Wyoming, ranged from 1.18 to 3.26 years. Cross sections surveyed showed that the discharge which filled the channel to the level of the floodplain, i.e., bankfull discharge, was nearly equal to the effective discharge suggesting that the streams adjusted to their effective discharge (Andrews, 1979).

Other studies have estimated the bankfull discharge frequency at approximately 1.5 to 2 years (Leopold, 1968). For changes in the regimen of flow, whether through land use or other changes, Leopold (1968) hypothesized that stream channels tend to react by an amount sufficient to maintain a similar quasiequilibrium under an altered flow regimen; thus the amount of channel enlargement would be roughly proportional to the increase in the 1.5-year flood (Hammer, 1979). The amount of channel enlargement resulting from a given level of urbanization is significantly related to topographic characteristics of the watershed. The most critical determinant appears to be basin slope. The process of channel enlargement is where adverse impacts may result on stream habitat and aesthetics (Hammer, 1972, and Arnold, 1996). The process is neither immediate nor eternal.

Most studies identifying impacts of urbanization on the frequent (mean) floods have been in humid areas where rainfall is gentle and vegetation is lush like the forests of King County Washington (Booth, 1997) or the Piedmont areas of Maryland (Klein, 1979) or Pennsylvania (Leopold, 1968). In these areas, rainfall intensities are generally lower than the rate at which the soil can absorb it. The rainfall can infiltrate where it first lands, then moves downslope as subsurface flow. Areas with a primarily subsurface hydrologic response are significantly impacted by the imperviousness of urbanization because the runoff response mechanism has changed from subsurface to overland (Booth, 1991).

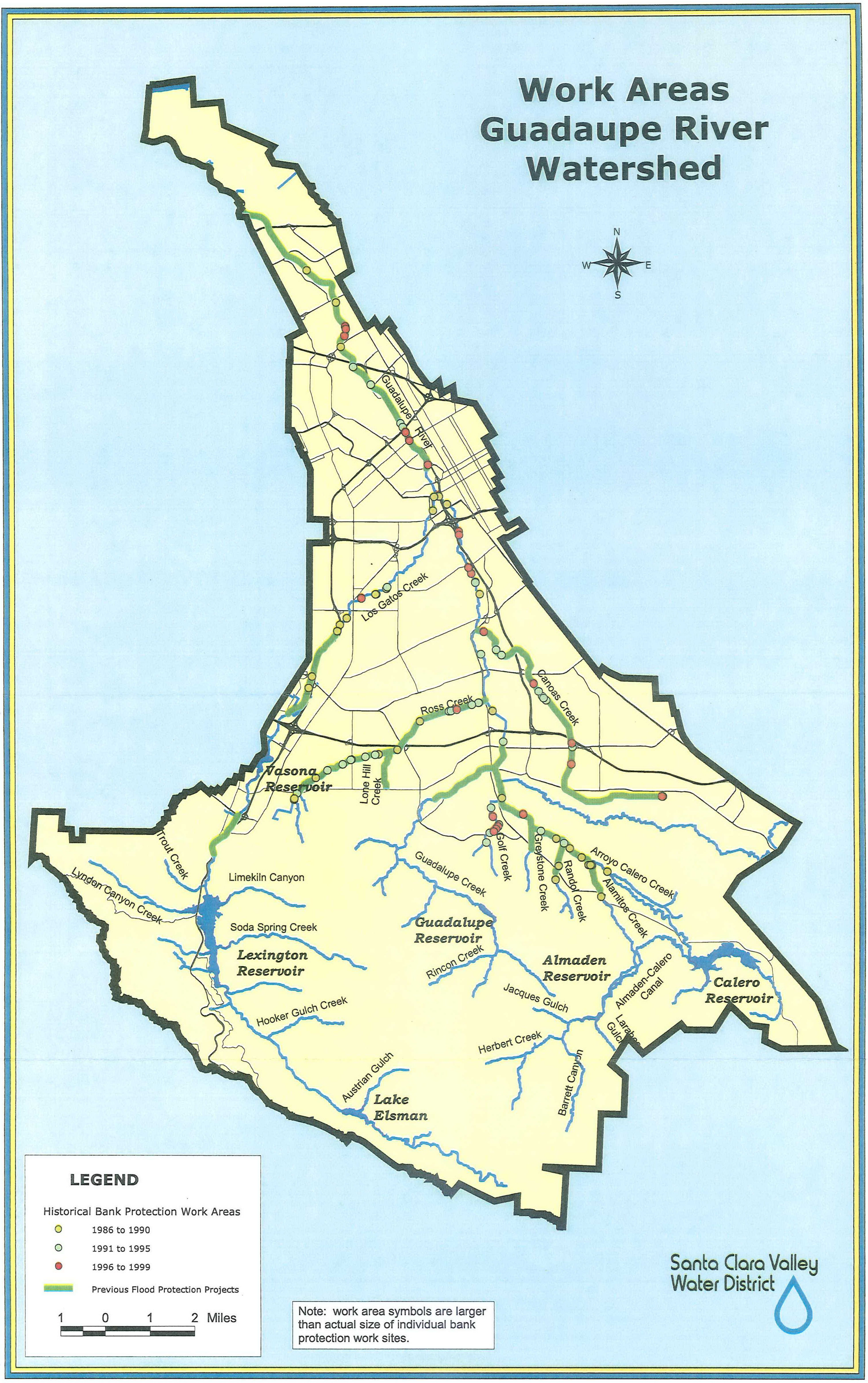
Less is known about the hydrologic impacts in semi-arid areas like the Santa Clara Valley. Typically, runoff is characterized by surface runoff where water moves quickly to the channel and all parts of the watershed contribute. "Semi-arid areas typically lack the vegetation cover, mature soil profile, and humic layer that generally intercept and store precipitation, enhance infiltration, and attenuate storm runoff in areas of moist climate" (Osterkamp, 2000). Flash flood watches and warnings are a familiar occurrence in Santa Clara Valley. "Where overland flow predominates, much of the precipitation reaches the channel under all storm conditions, regardless of the level of urban development" (Booth 1991).

The concept of geomorphological work and its relation to an effective discharge, which were applied first to work accomplished by mean annual floods of humid landscapes, in semi-arid regions appear to relate to increasingly greater and infrequent rainfall-runoff events as measures of mean precipitation decrease (Osterkamp, 2000). Similar findings have been made for small arid watersheds (Baker, 1977).

With the above understanding, we can focus on areas where changes of the hydrologic regime could potentially disrupt the stream's quasiequilibrium and cause erosion and the loss of riparian habitat. Streams without riparian vegetation and streams that have been previously modified by construction

(Figure 2.1) would not be susceptible to adverse impacts to riparian habitat caused by changes in the hydrologic response.

In this study, we have first analyzed those factors which may have had a significant impact on the hydrologic response of the watershed. Then the erosion and sedimentation problems which have been previously identified were analyzed for possible causes. This study also presents a summary of previous investigations of flood flow reduction in the watershed and finally additional storage alternatives to mitigate urban flows were developed.



Work Areas Guadaupe River Watershed

Note: work area symbols are larger than actual size of individual bank protection work sites.

Figure 2.1

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III. ANALYSIS OF SIGNIFICANT HISTORIC IMPACTS TO THE GUADALUPE RIVER HYDROLOGY

There are at least two factors which have altered the hydrologic response in the Guadalupe River watershed: reservoir construction and conversion of land use due to urbanization. We have analyzed the significance of each factor compared to an unaltered, undeveloped base condition, utilizing the District's hydrology methodology, and the District's existing HEC-1 hydrologic model. The hydrographs in the HEC-1 model have a slightly greater peak flow than the Corp's hydrographs being used for design.

UNDEVELOPED CONDITION (RURAL)

The District utilizes a regional regression and correlation methodology to estimate design flood flows and their frequency of occurrence (Saah, A., 1976; Wang et al., 1998). All the unregulated streamgage records within Santa Clara County and adjacent region were analyzed and cross-correlated to extend the length of records. The correlation and extension of each station's records provided a more uniform base upon which to perform regressions. The physical characteristics of each watershed, such as drainage area and Mean Annual Precipitation, were regressed with the statistical parameters from each streamgage record analysis, such as mean and standard deviation. The result is a flood frequency predictive model for all natural unregulated watersheds. The regression model was applied to the subwatersheds in the Guadalupe River basin. Runoff hydrographs were developed, combined, and routed downstream using the HEC-1 hydrologic model. The two hydrographs, for the 1 percent and 10 percent events, in Figures 3.1 and 3.2 represent a natural, undisturbed, predevelopment, unregulated watershed for the Guadalupe River immediately downstream of the Los Gatos Creek confluence and immediately downstream of the Alamitos Creek confluence.

WITH RESERVOIRS

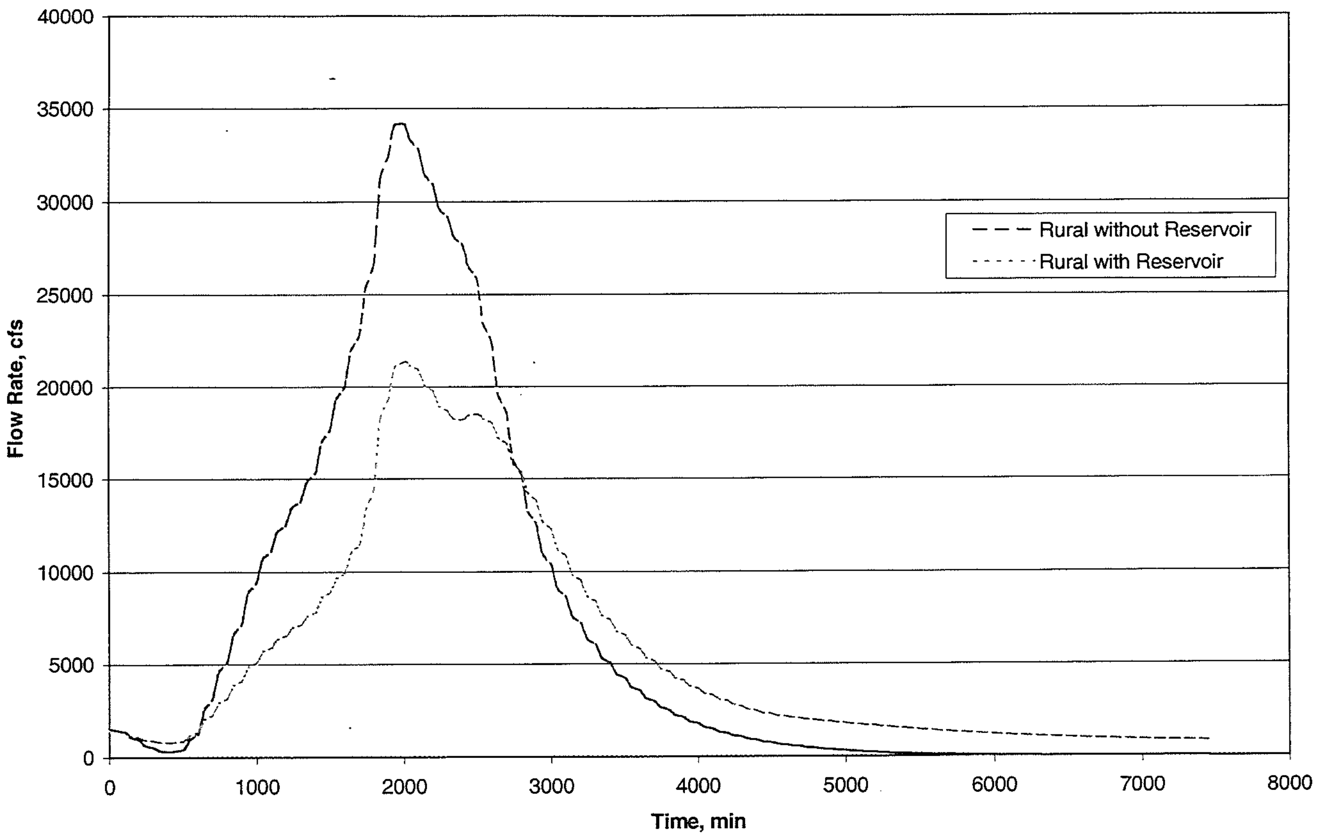
There has been extensive reservoir development in the Guadalupe watershed. Four major dams have been constructed by the District beginning in the 1930s: Almaden, Guadalupe, Calero, and Lexington (Lenihan). San Jose Water Company has constructed and operates Lake Elsman located above Lenihan dam. Reservoirs tend to attenuate peak flows and lengthen the hydrographs because of the coincidental storage capacity which may be available at the beginning of a storm and because of the additional surcharge storage which is necessary to discharge flow over the reservoir spillways. Figures 3.1 and 3.2 also show the hydrographs, as they have been affected by the four reservoirs. Frequency curves based on the 1 percent and 10 percent peak flows are shown on Figures 3.3 and 3.4. The frequency curve demonstrates how dramatically the construction of the reservoirs has reduced all flood flows relative to the natural, unregulated watershed. There is over a 50 percent reduction in the mean flow.

WITH URBAN DEVELOPMENT

In 1976, the District completed a study of the hydrologic effect of the urban development which has recurred in Santa Clara County (Saah, 1976). The study focused on Ross Creek in the Guadalupe River Watershed and found that the hydrologic response was most affected by the degree of imperviousness and the capacity of the storm drain system. Urban development has been relatively uniform throughout the valley and typically includes roadways, rooftops, sidewalks, and other impervious surfaces and curbs and gutters and storm drains to carry the runoff to the creeks. Storm

Figure 3.1

Guadalupe River D/S Los Gatos Creek 100-Yr



Guadalupe River D/S Los Gatos Creek 10-Yr

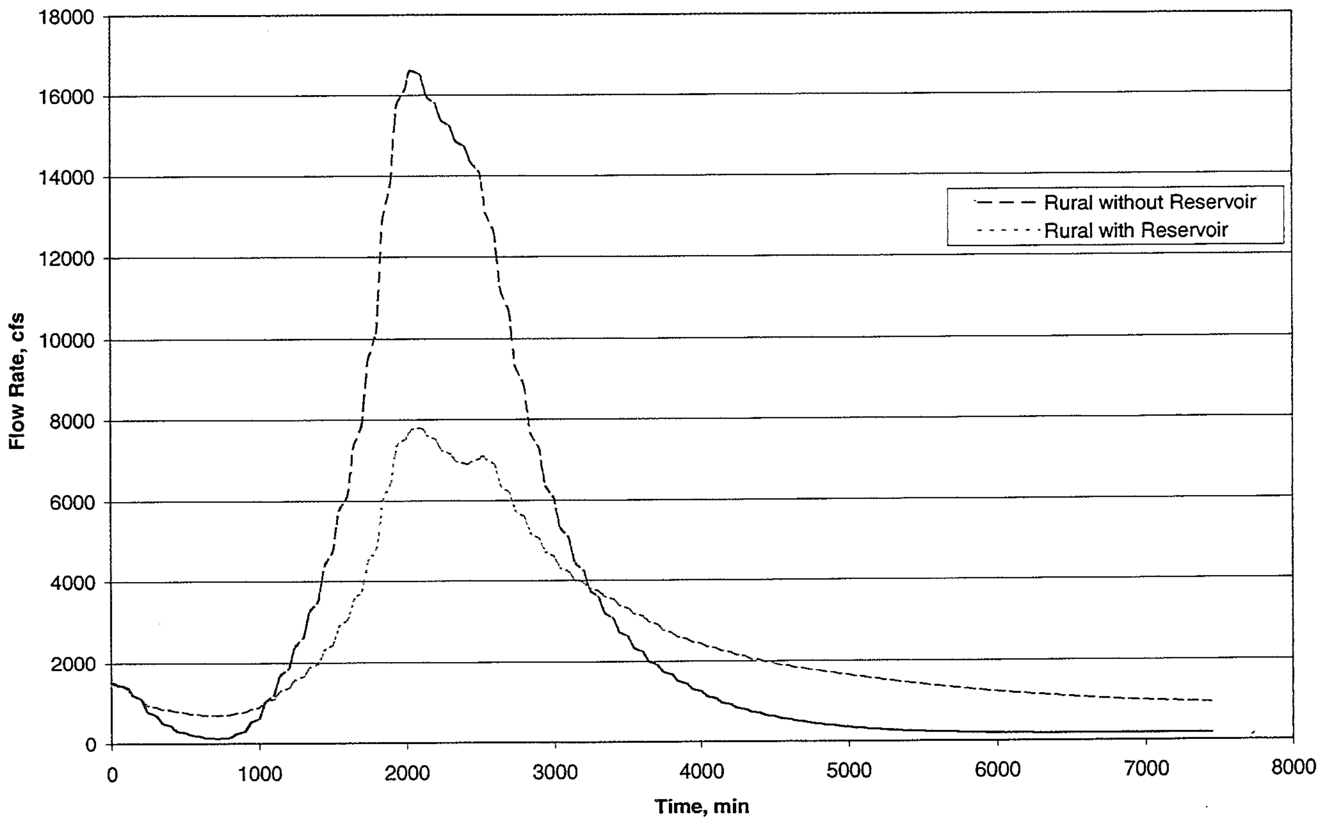
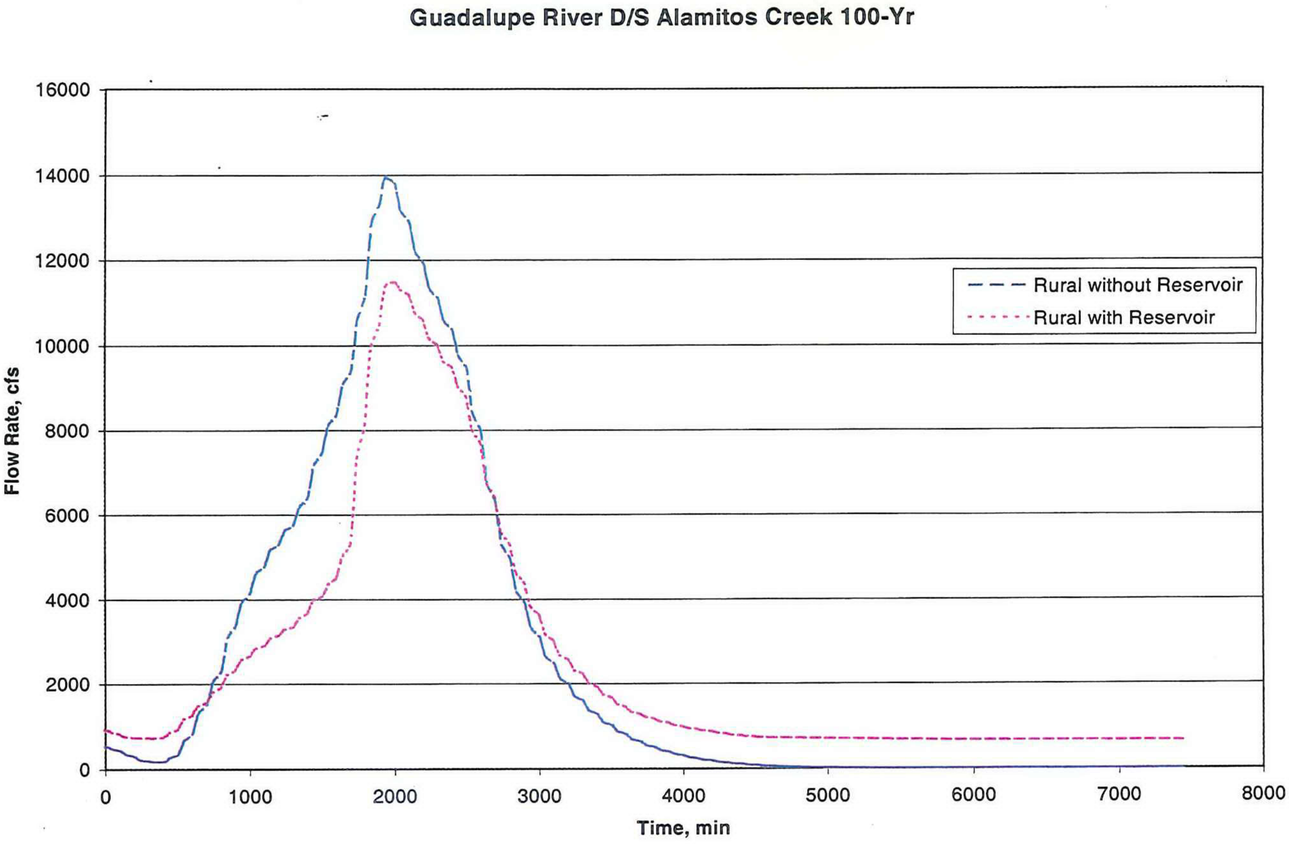


Figure 3.2


Guadalupe River D/S Alamitos Creek 100-Yr
Flow Rate, cfs
16000
14000
12000
10000
8000
6000
4000
2000
0
0
1000
2000
3000
4000
5000
6000
7000
8000
Time, min
Rural without Reservoir
Rural with Reservoir

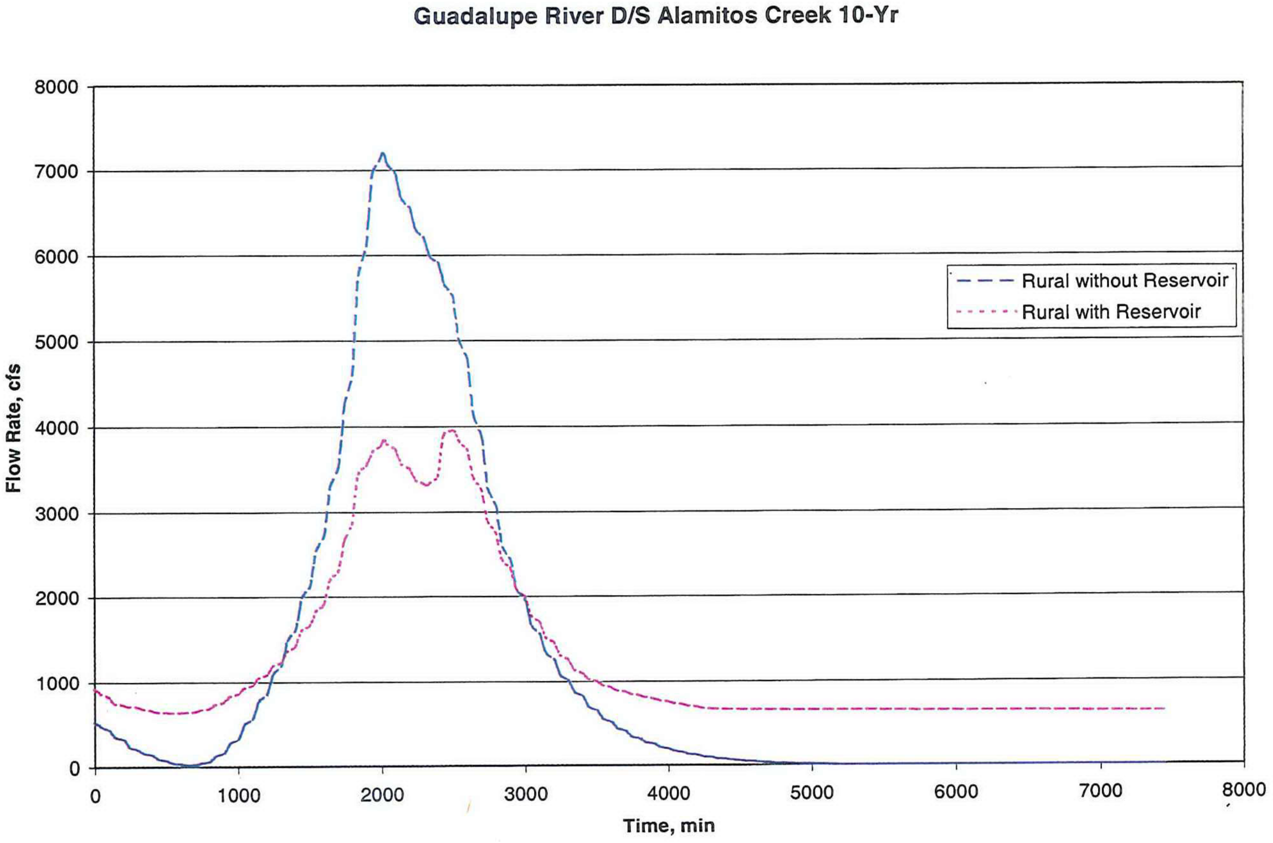

Guadalupe River D/S Alamitos Creek 10-Yr
Flow Rate, cfs
8000
7000
6000
5000
4000
3000
2000
1000
0
0
1000
2000
3000
4000
5000
6000
7000
8000
Time, min
Rural without Reservoir
Rural with Reservoir

Figure 3.3

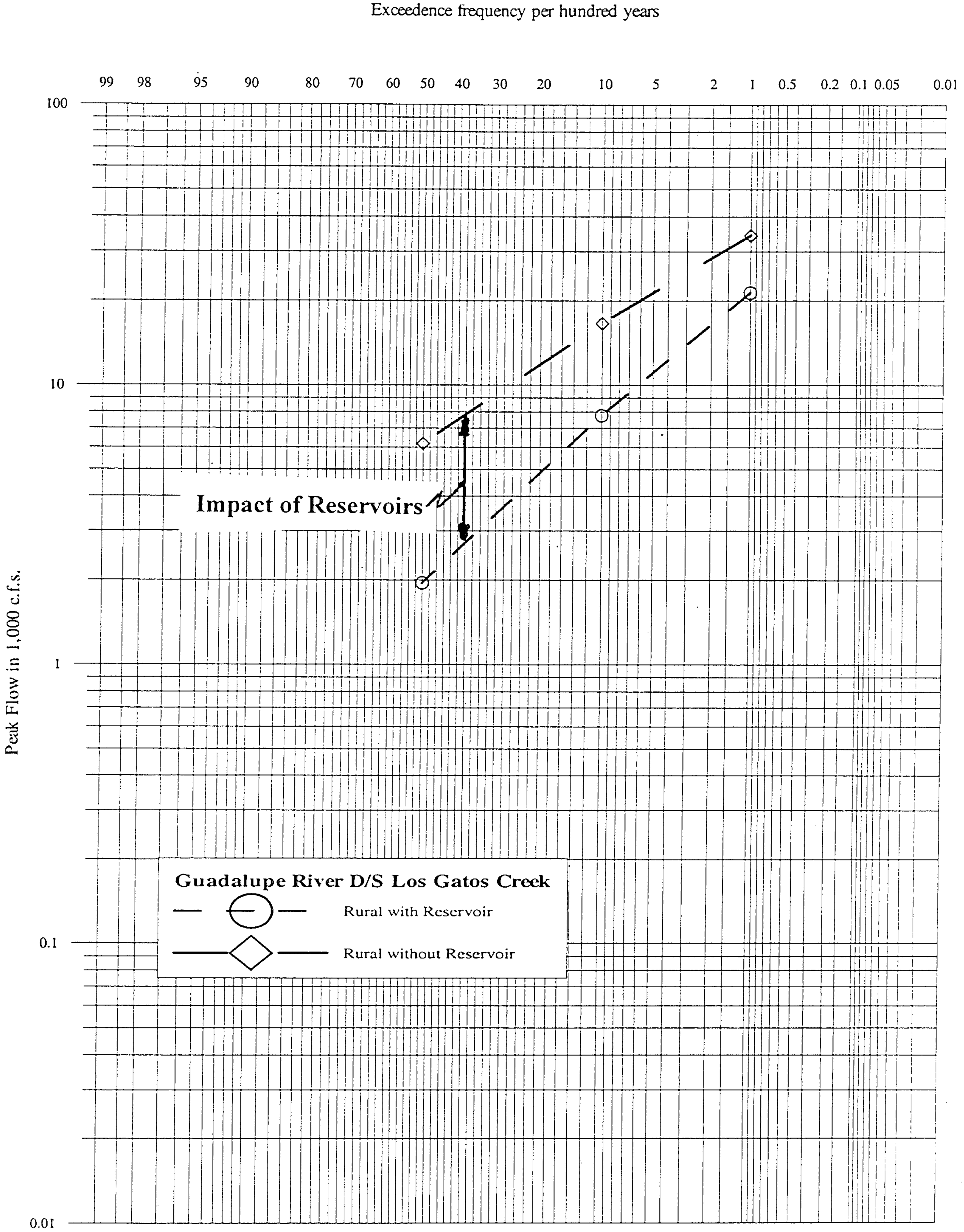
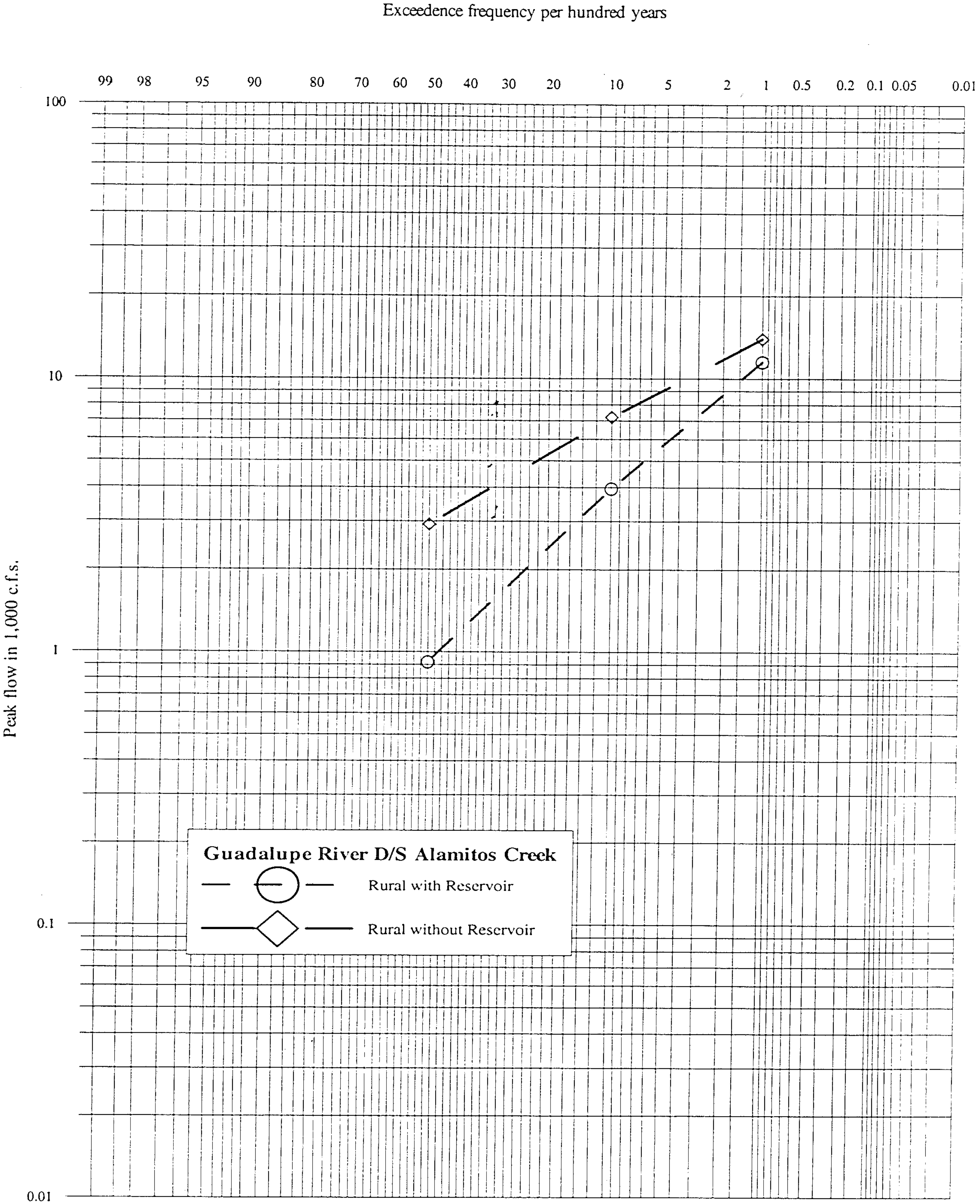

Exceedence frequency per hundred years
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01
100
10
1
0.1
0.01
Peak Flow in 1,000 c.f.s.
Impact of Reservoirs
Guadalupe River D/S Los Gatos Creek
Rural with Reservoir
Rural without Reservoir

Figure 3.4


Exceedence frequency per hundred years
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01
100
10
1
0.1
0.01
Peak flow in 1,000 c.f.s.
Guadalupe River D/S Alamitos Creek
Rural with Reservoir
Rural without Reservoir

drains have been designed to carry frequent events, for example 3–20 year, with the extreme events allowed to pond temporarily in the streets or other low spots where damage would be avoided.

The District study found that the most frequent floods (events more frequent than 5 to 10 years return period) were increased in magnitude by urbanization, reflecting the effectiveness of the urban drainage systems for small storms (Figure 3.5). The urban effect on the more extreme events on Ross Creek was much less significant because the conditions necessary to produce these extreme runoff events in natural watershed, such as saturated watersheds and high runoff rates, are similar to the conditions created with urban development. The lesser impact of the urban area for larger events is consistent with others findings (Crippen, 1965; Hollis, 1975; Leopold, 1968; and Branson, 1981). In the Santa Clara Valley, the typical storm drain system is not designed with sufficient pipe capacity to discharge the full flow directly into the creek. The limitations of the pipe capacity force the runoff from extreme events into temporary storage in streets and low-lying areas. This is observable throughout the valley floor during intense rainfall. The storm drain capacity, as a critical factor in the hydrologic response, was the significant discovery in the District's study which showed a reduction in flood flow for events greater than 10 years (Saah, 1976).

This urban hydrology model was applied to the Guadalupe River watershed for the existing urban development condition, represented by the City of San Jose's Horizon 2000 Plan, and a future urbanization condition represented by the Horizon 2020 plan. The impact of the two urban conditions is shown in Figures 3.6 and 3.7 in comparison with the hydrographs representing a natural watershed. The hydrographs show the attenuation for the larger events.

The urbanization of the watershed has been added to the flood flow exceedence frequency curves in Figures 3.8 and 3.9 relative to the rural watershed and the influence of the reservoirs. There is very little hydrologic difference between the year 2000 urban area and the year 2020 plan. The urban watershed (Horizon 2000) represents the current hydrologic condition.

The most significant hydrologic change on flood flow in the watershed has been the reduction in peaks caused by the reservoirs. The reservoirs' effect on flow has been to reduce flood peaks and the "bankfull" flow and reduce the stresses potentially causing erosion and the loss of riparian habitat. The District's urban analysis shows no change in the hydrologic range that could potentially disrupt the stream's quasiequilibrium for a semiarid area and cause erosion and the loss of riparian habitat.

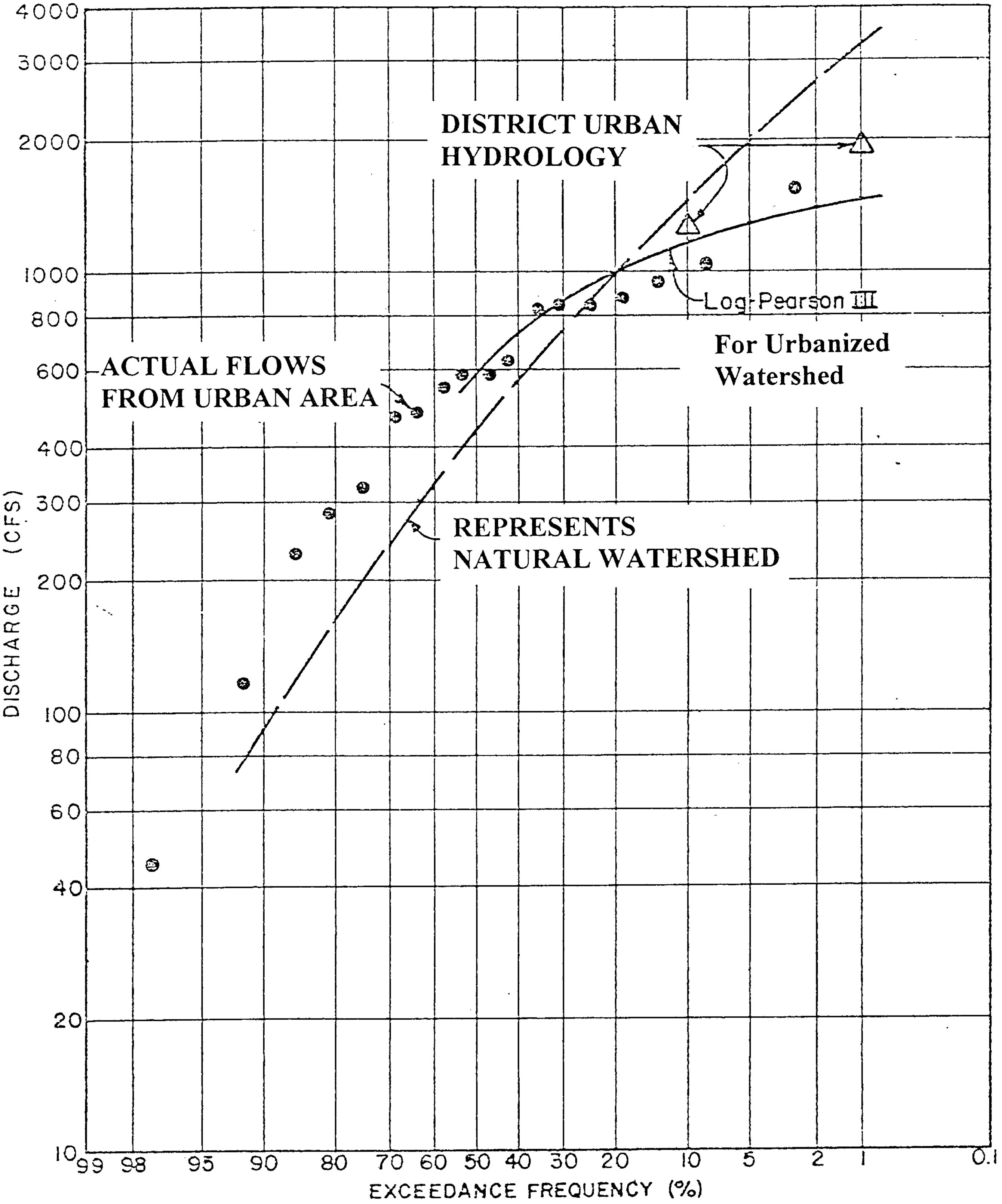
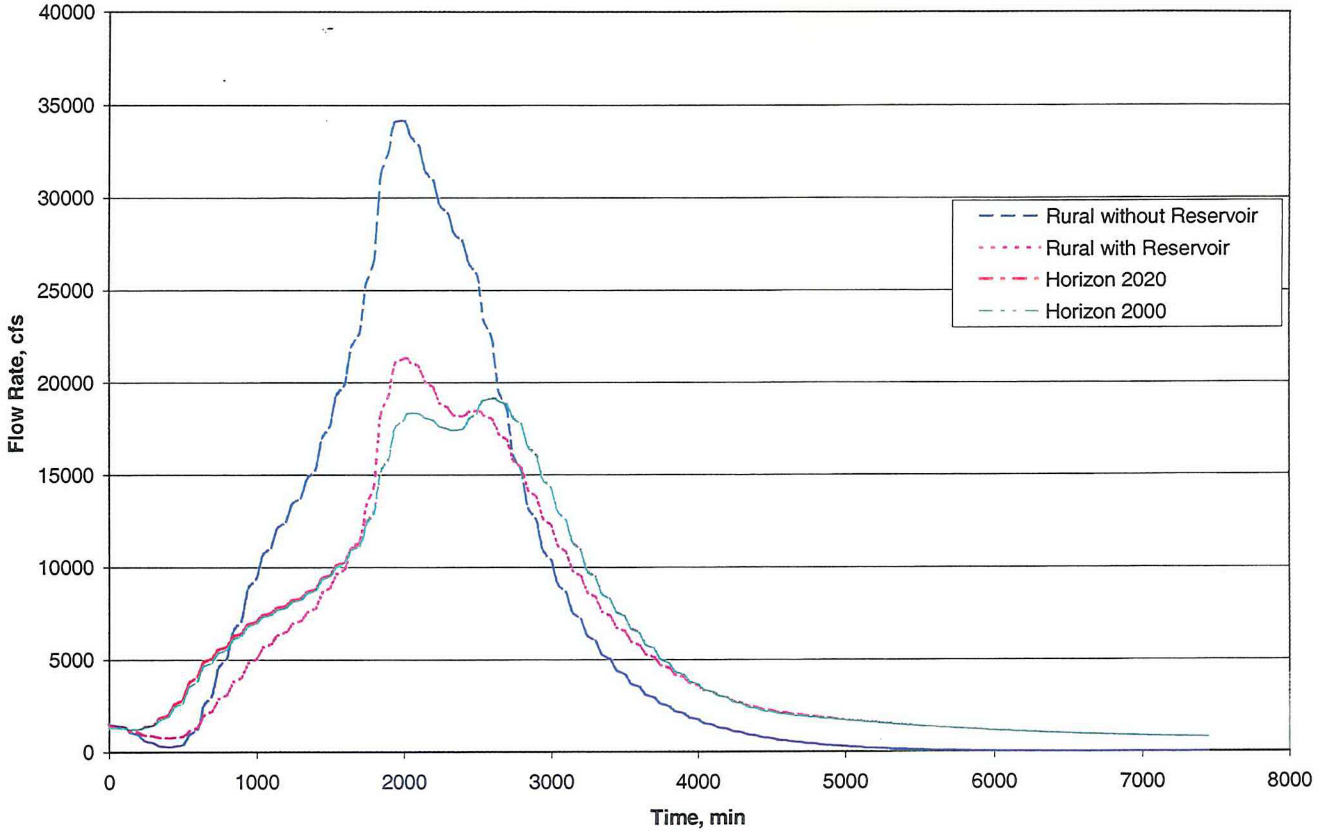


FIG. 3.5 ROSS CREEK AT STREAMGAGE # 51

Figure 3.6


Guadalupe River D/S Los Gatos Creek 100-Yr

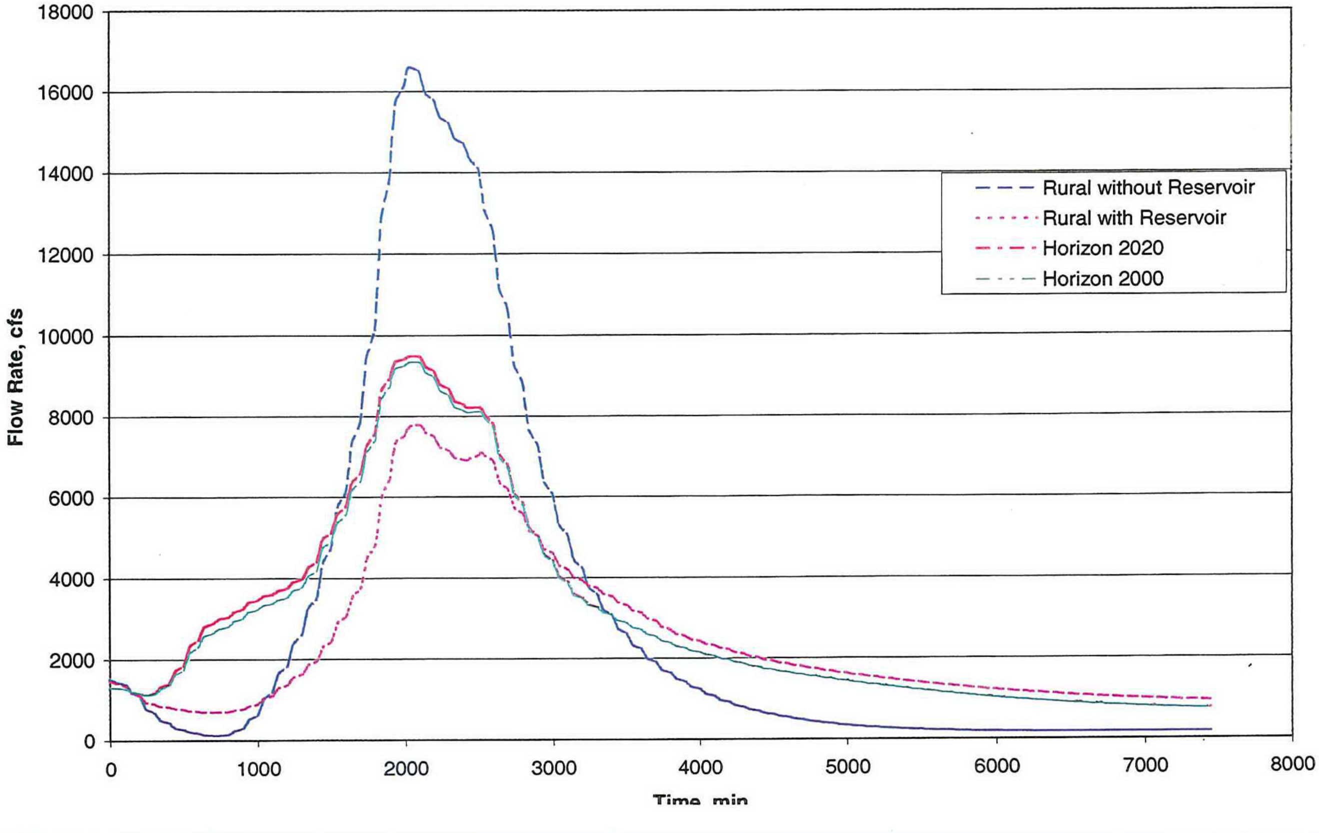
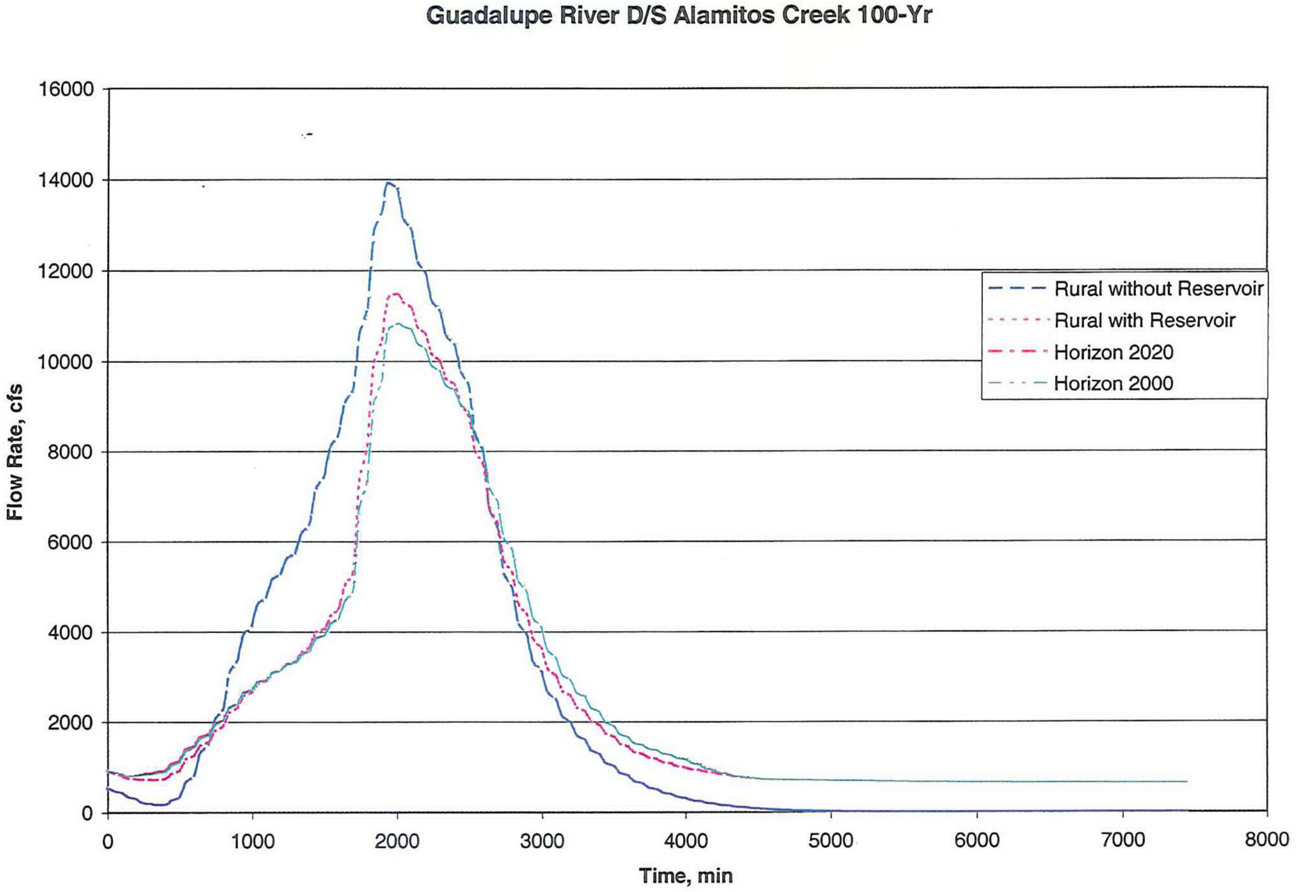

Guadalupe River D/S Los Gatos Creek 10-Yr

Figure 3.7


Guadalupe River D/S Alamitos Creek 100-Yr
Flow Rate, cfs
16000
14000
12000
10000
8000
6000
4000
2000
0
0
1000
2000
3000
4000
5000
6000
7000
8000
Time, min
Rural without Reservoir
Rural with Reservoir
Horizon 2020
Horizon 2000

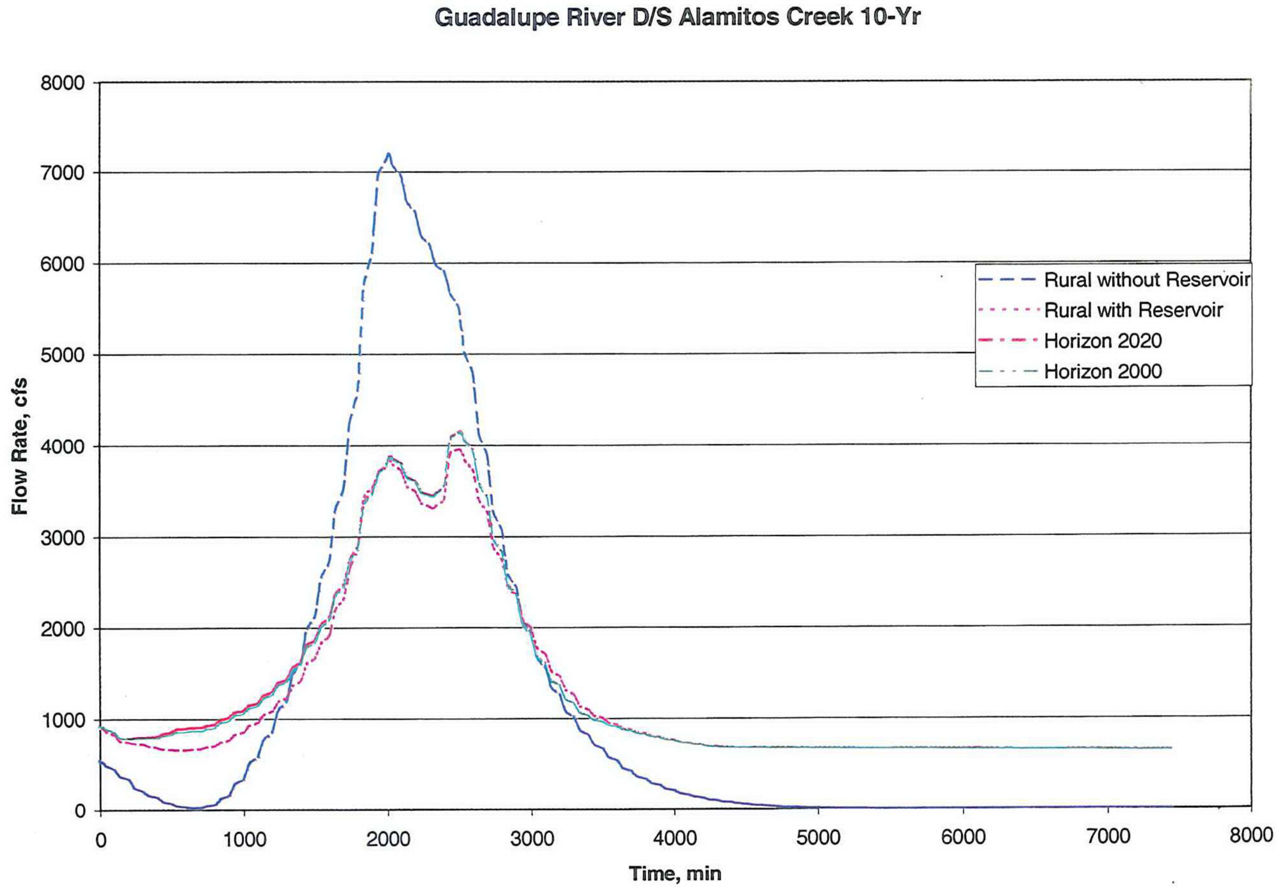
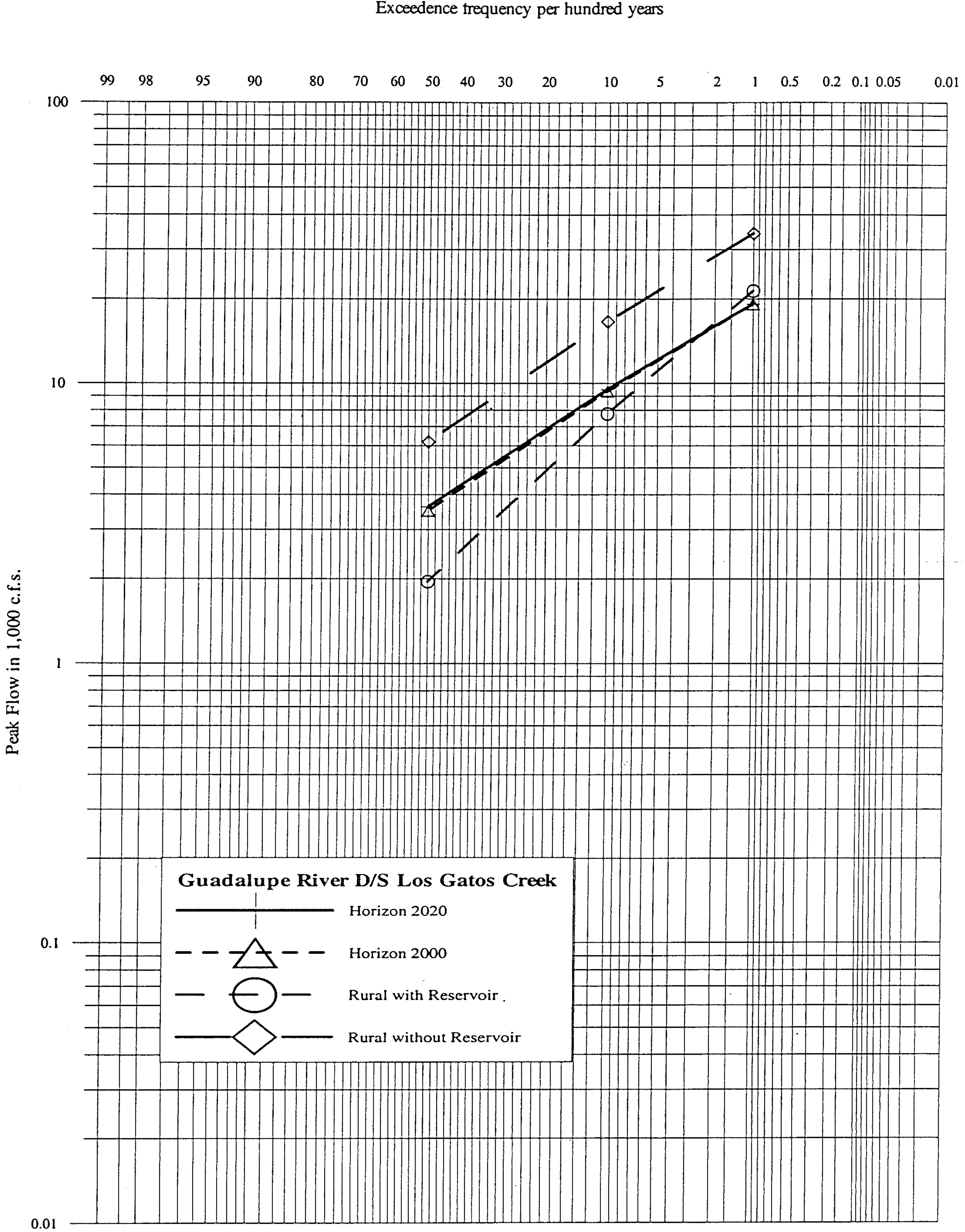

Guadalupe River D/S Alamitos Creek 10-Yr
Flow Rate, cfs
8000
7000
6000
5000
4000
3000
2000
1000
0
0
1000
2000
3000
4000
5000
6000
7000
8000
Time, min
Rural without Reservoir
Rural with Reservoir
Horizon 2020
Horizon 2000

Figure 3.8



Exceedence frequency per hundred years
99
98
95
90
80
70
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40
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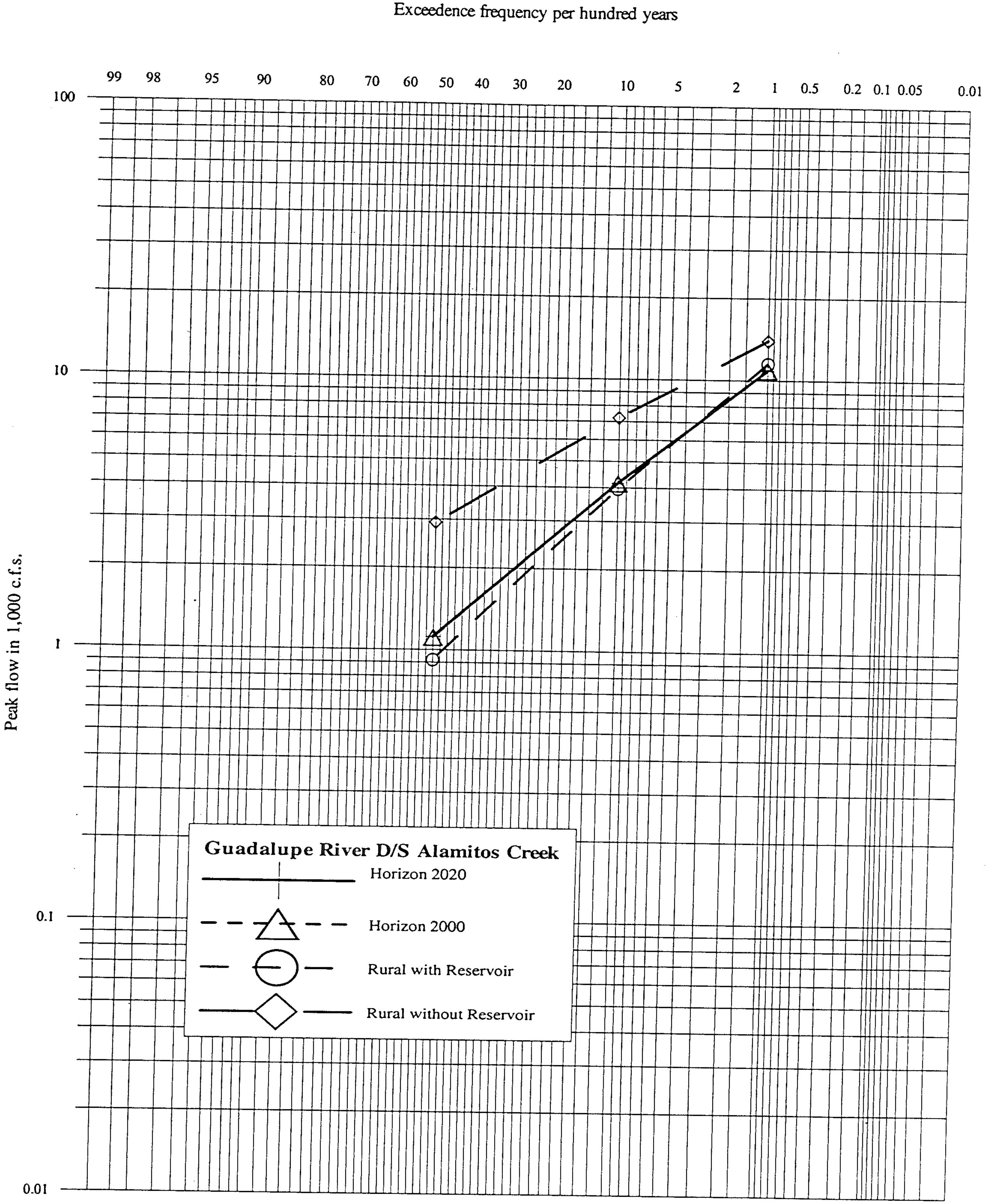
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01
100
10
1
0.1
0.01
Peak Flow in 1,000 c.f.s.
Guadalupe River D/S Los Gatos Creek
Horizon 2020
Horizon 2000
Rural with Reservoir
Rural without Reservoir

Figure 3.9


Exceedence frequency per hundred years
99
98
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90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01
100
10
1
0.1
0.01
Peak flow in 1,000 c.f.s.
Guadalupe River D/S Alamitos Creek
Horizon 2020
Horizon 2000
Rural with Reservoir
Rural without Reservoir

IV. EROSION PROBLEMS

The District and others have experienced numerous erosion problem areas on nearly every stream in the county. Streambank erosion can threaten the integrity of our flood protection facilities, threaten loss or damage to adjacent properties, and cause loss of streamside habitat. Figure 4.1 is based on the District's recently-adopted Stream Maintenance Program Environmental Impact Report. It shows the District's streambank repair sites over the past 15 years.

The bank failure or erosion at these sites has resulted from a variety of causes. Not all involve erosion of soil. Several sites are repairs of channel lining in projects which are aging. Many sites are in facilities constructed early in the District's flood protection program before there was adequate design criteria. Many erosion problems resulted from encroachment by adjacent property owners or improper bridge construction. There were also several repairs where the streams have been actively cutting into the noncohesive alluvial soils.

REPAIR OF CHANNEL LINING

Several sites, such as on Hale and Permanente, are shown in Figure 4.1 for the repairs to the failure of concrete lining constructed by the District in the 1950s and 1960s. The District did a comprehensive investigation on Permanente Creek and found seven distinct patterns of deterioration or deficiency due to a combination of factors including age, construction details, and channel type (Biggs Cardosa, 2000). Two of these deficiencies posed a structural stability concern: excessive wall deflections and longitudinal cracking of the invert slab.

INADEQUATE DESIGN CRITERIA

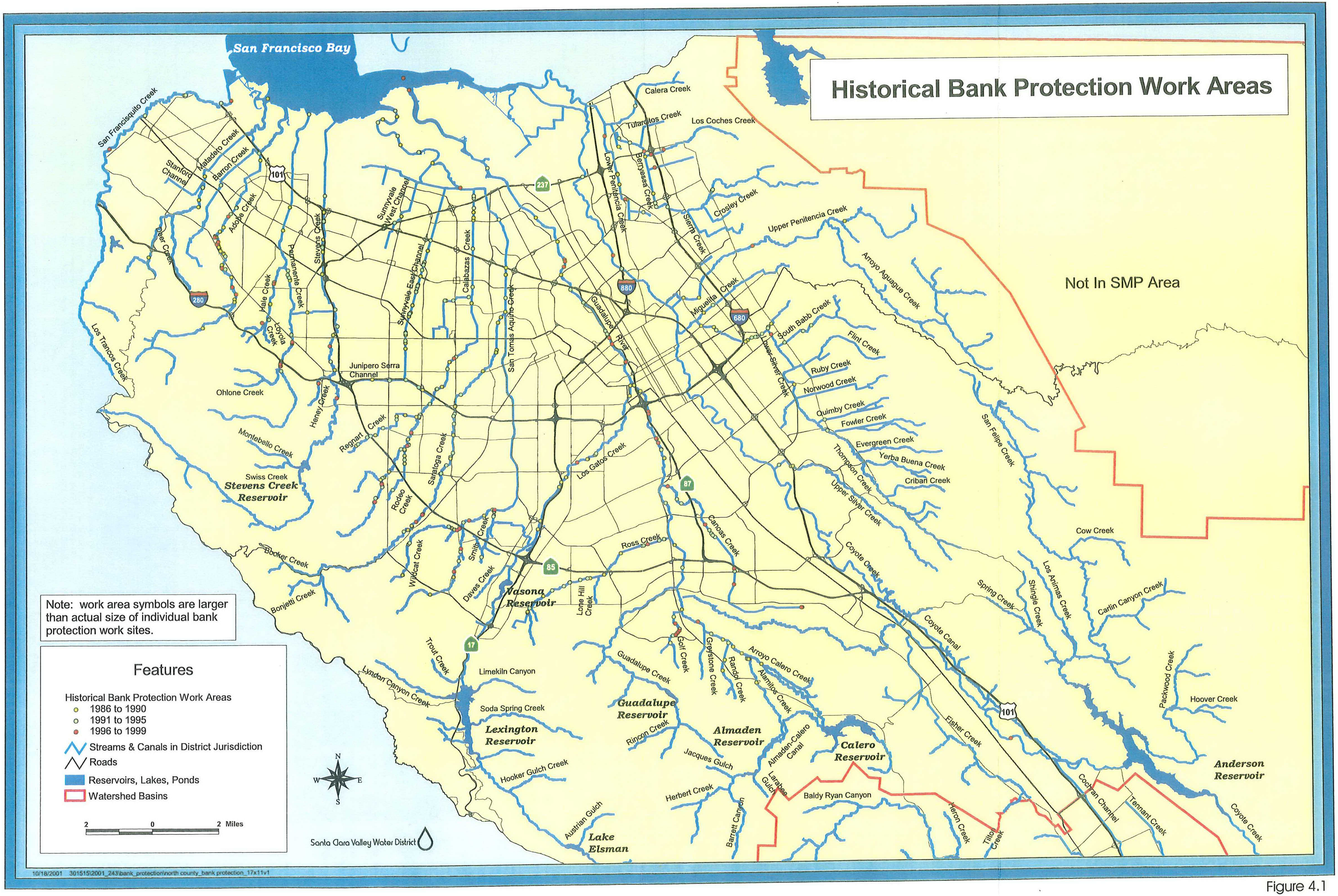
Many of the District's early flood protection projects consisted of enlarging and/or realigning the natural or previously-altered creek, removing any vegetation, and constructing a trapezoidal earthen or lined channel.

Ross Creek, Calabazas Creek, and many other projects have experienced ongoing deterioration in unlined sections because the 8 feet per second velocity used for design produced excessive tractive forces (Woodward-Lundgren Associates, 1971).

Resistance to tractive forces is developed differently in cohesive and noncohesive soils. In cohesionless soils, erosion or particle removal occurs when the tractive forces created by flowing water are sufficient to lift the particle and transport it downstream. Resistance to the erosion, therefore, is provided by particle weight and interparticle friction. Usually erosion of cohesionless soils occurs in the form of sheet-type soil removal in long uniform strips.

Cohesive soils provide resistance to erosion by virtue of their cohesive strength. Interparticle bonding of the clay materials provides this strength. Erosion of clay materials occurs when the interparticle bond forces (cohesion) have been exceeded or weakened. The particles then become suspended in colloidal forms. Particles in overconsolidated clays at natural moisture contents normally are held in place by relatively high interparticle forces and consequently offer high resistance to tractive forces.

Historical Bank Protection Work Areas



Note: work area symbols are larger than actual size of individual bank protection work sites.

Features

Historical Bank Protection Work Areas
- ○ 1986 to 1990
- ○ 1991 to 1995
- ● 1996 to 1999

Streams & Canals in District Jurisdiction
Roads
Reservoirs, Lakes, Ponds
Watershed Basins

10/18/2001 301515\2001_243\bank_protection\north county_bank protection_17x11v1

Figure 4.1

ENCROACHMENTS

Historical encroachments into the flow way, typically by adjacent property owners or improper bridge construction, have caused many erosion problems. A significant artificial constriction on Adobe Creek at Edith Avenue created a high velocity flume, which tore out walls downstream and caused sediment deposition upstream (Adobe, 1999). The District investigated solutions to the ongoing erosion in this reach with the property owners. The District Board adopted the EIR for a three-staged channel enlargement in May 1999. The existing channel cross section and the District's proposed solution are shown in Figure 4.2.

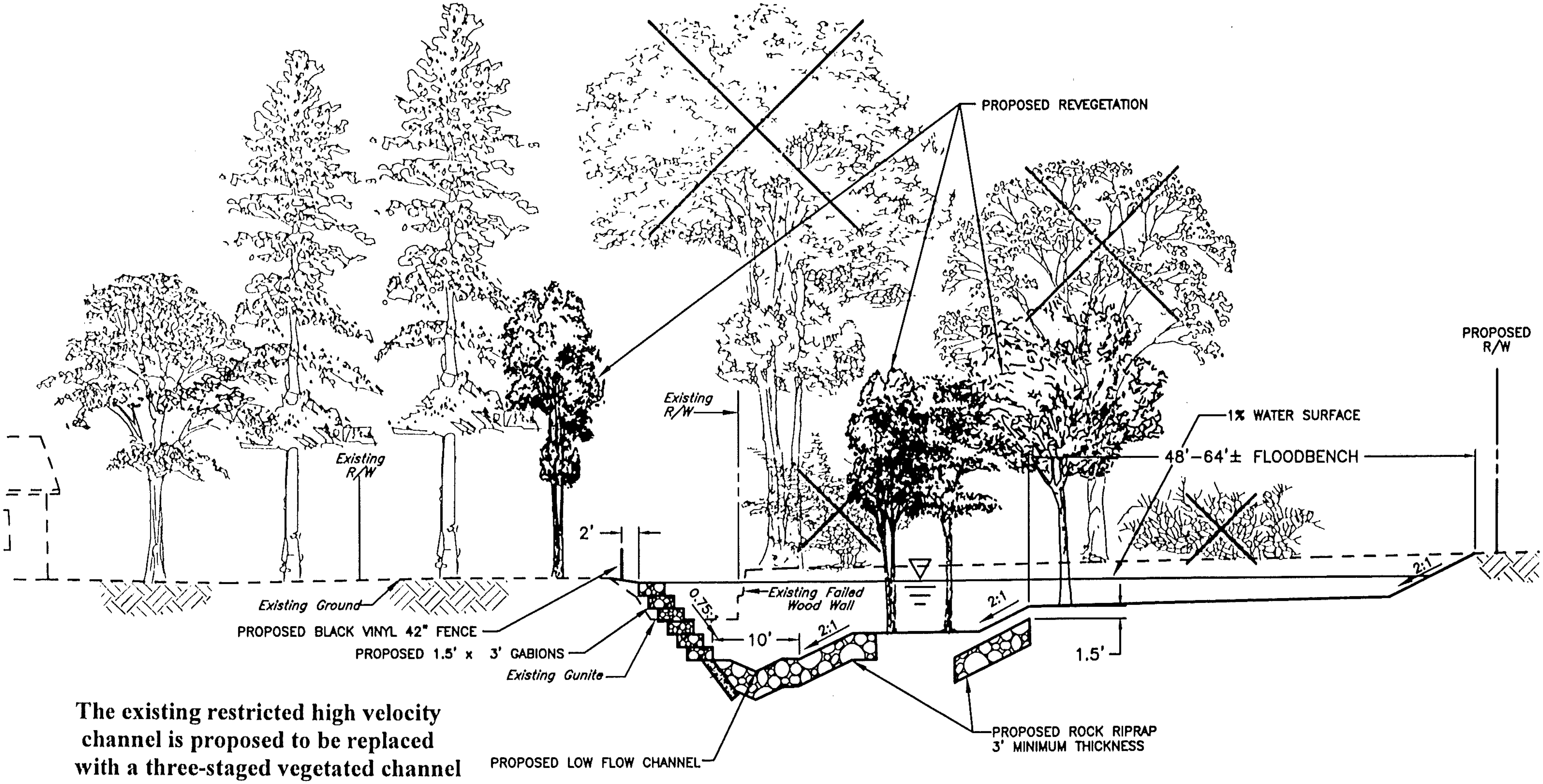
Culverts constructed above the natural creek profile have also caused sediment deposition upstream and severe erosion problems downstream, as shown in Figure 4.3 on Adobe Creek at O'Keefe Lane. This is one of the problem areas identified to be corrected by the Adobe Creek project (Adobe, 1999). A larger culvert with three small drops is proposed to reduce the erosive force downstream and reestablish a natural grade for the creek upstream of the culvert.

NONCOHESIVE ALLUVIUM

In December 1867, Los Gatos Creek overflowed during a severe flood and changed course to its present alignment (James and McMurry). The old creek bed, still visible on topographic maps (Figure 4.4), eventually became Dry Creek Road. Since the change in course, the creek has actively cut through the noncohesive alluvial materials to the extent that many of the creek banks are now over 20 feet high and near vertical. Construction of Lexington Dam has attenuated the flood flows and substantially reduced the erosive stress but the noncohesive soil continues to require remedial repairs. Shifting alignments and moving noncohesive materials is characteristic of the creeks draining the hills around the valley. San Francisquito and Upper Penitencia are other examples of creeks, with relatively small watershed contribution from the urban valley floor, where there has been ongoing streambank repair needs (Figure 4.1). Alamitos Creek, where flows predominately originate from the upper watersheds and are attenuated by two reservoirs, also has ongoing erosion problems as was shown (Figure 2.1). The District's flood protection improvements on Alamitos Creek consisted only of set-back levees and bypass channels. They did not disturb the natural creek setting.

IMPACT OF UPPER GUADALUPE RIVER PROJECT

The Guadalupe River Watershed Planning Study identified active stream bank erosion as a problem (Cheong, 1999), particularly between Willow Street and Almaden Expressway (Reaches 7-10A). Large sections of stream bank had become saturated and slipped into the creek taking property and vegetation. Property owners have responded with a variety of measures including concrete rubble and wooden retaining walls. These problems are in areas where the upstream reservoirs have substantially reduced the effective discharge (Figures 3.8 and 3.9) and erosive stresses. One of the benefits of the proposed Upper Guadalupe River project is the further reduction of the forces causing erosion. The proposed bypass channel and the channel widening bench elevations are designed to begin operating at an approximately 1,500 cfs bankfull discharge. This maintains the sediment transport capability of the bankfull channel and creates a functional floodplain release at greater events which reduces the erosive stresses in the natural channel. The excavated bench for the

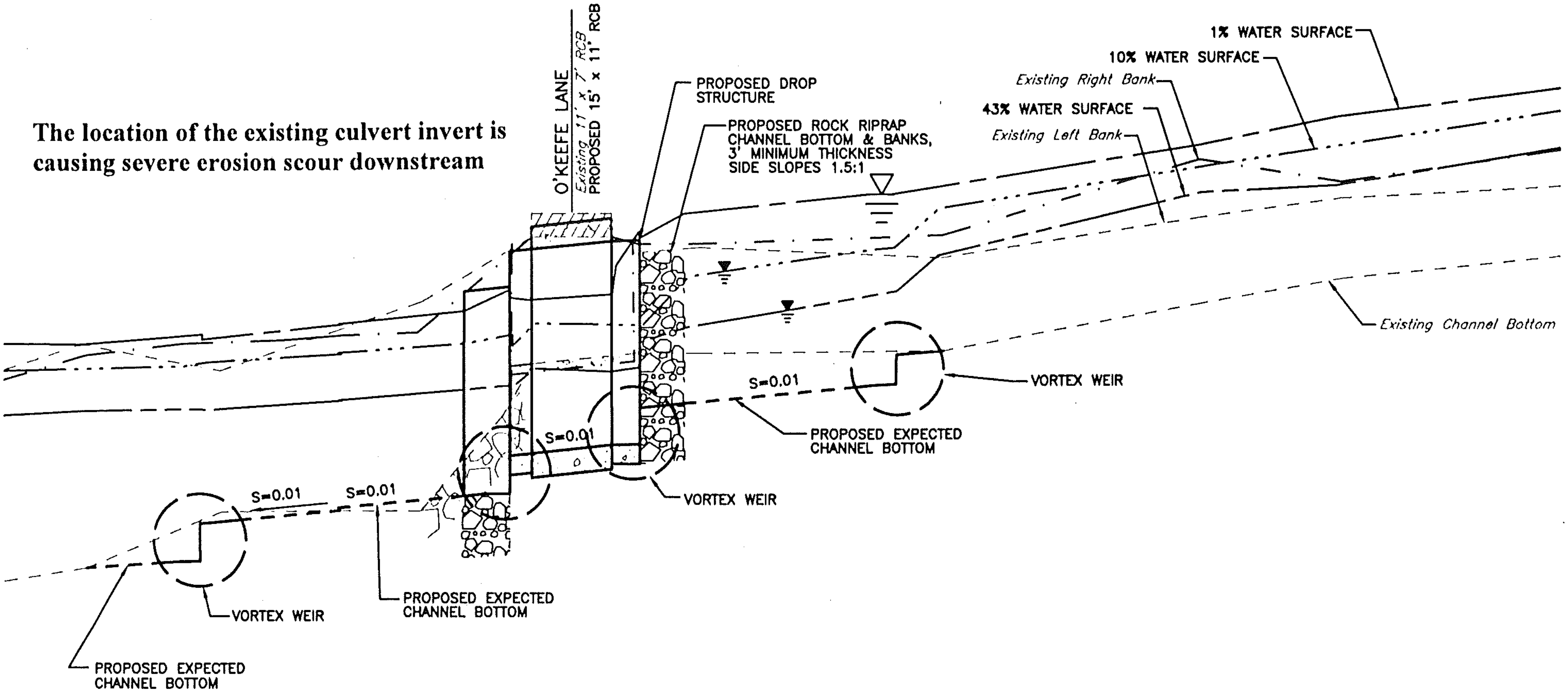


The existing restricted high velocity channel is proposed to be replaced with a three-staged vegetated channel

Adobe Creek Encroachment

Figure 4.2

DWG FILE NAME: G:\dwgs\adskproj\301515\Fig_1.DWG



The location of the existing culvert invert is causing severe erosion scour downstream

Adobe Creek Bridge Encroachment

Figure 4.3

DWG FILE NAME: G:\dwgs\sdskproj\301515\Fig_2.DWG

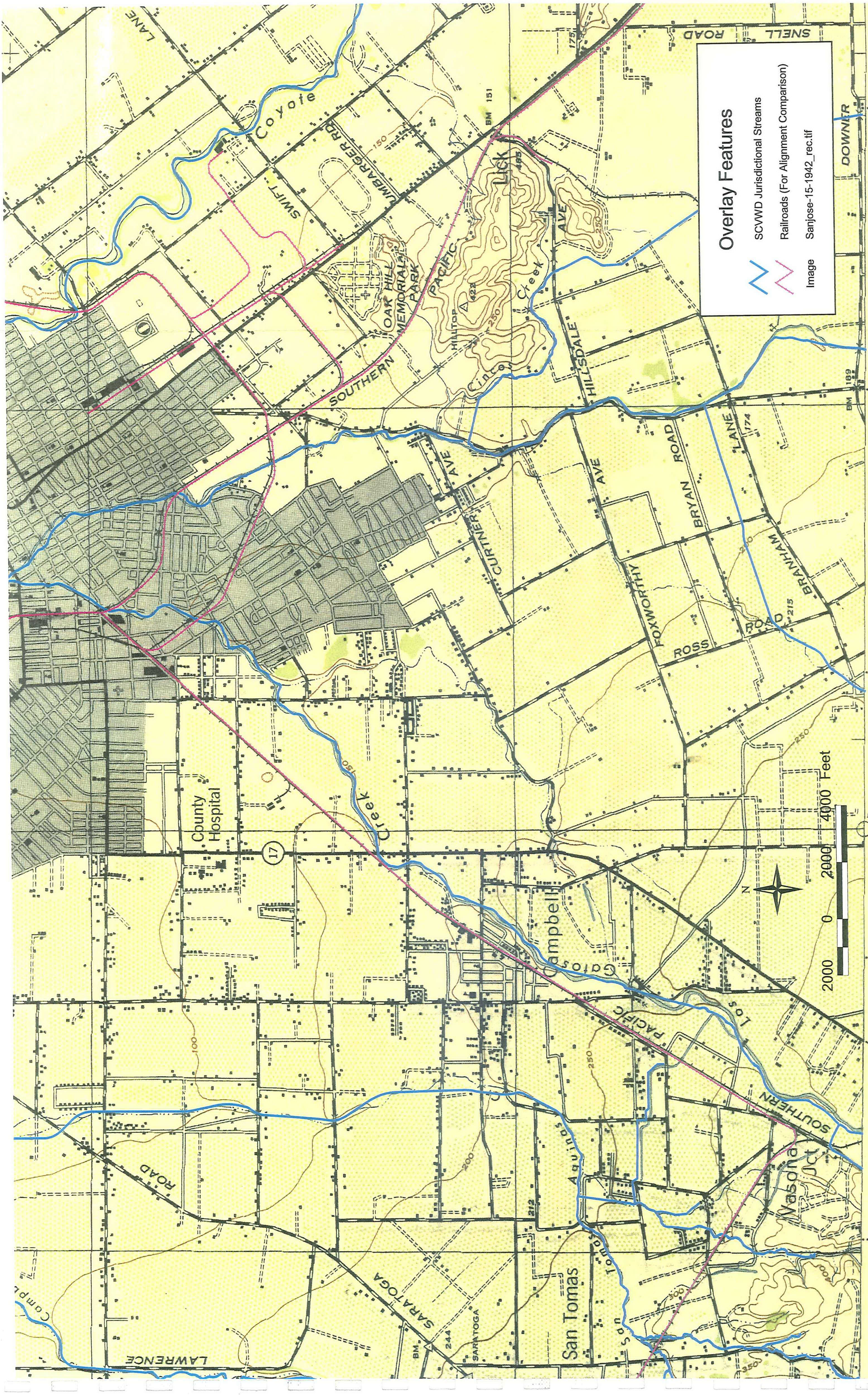
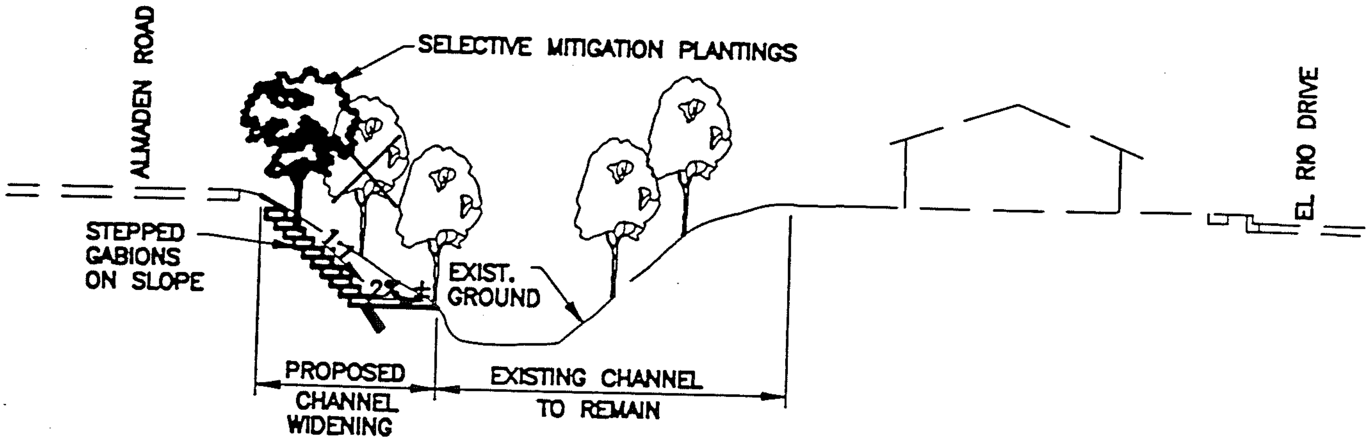
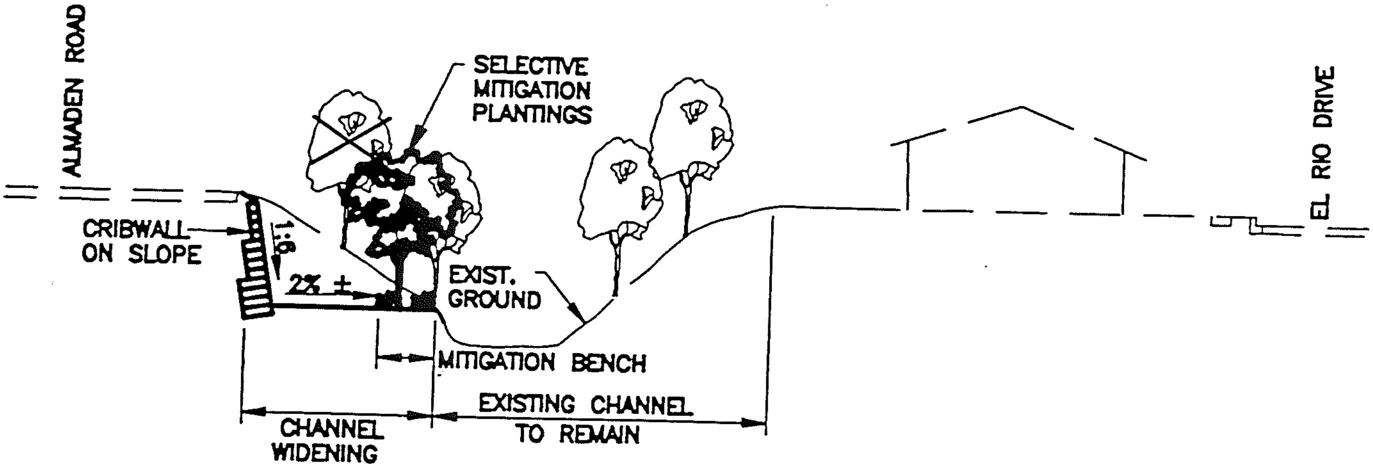

Overlay Features
SCVWD Jurisdictional Streams
Railroads (For Alignment Comparison)
Image
Sanjose-15-1942_rec.tif
Coyote
LANE
SNELL
ROAD
DOWNER
SWIFT
UMBARGER RD
BM 151
150
Lick
HILLSDALE
AVE
OAK HILL MEMORIAL PARK
SOUTHERN
PACIFIC
HILLTOP
Creek
CURTNER
AVE
FOXWORTHY
AVE
BRYAN
ROAD
BRANHAM
LANE
ROSS
ROAD
BM 189
174
215
250
(LOS GATOS)
County Hospital
17
Creek
Campbell
Los
Gatos
PACIFIC
SOUTHERN
Vasona Jct
San Tomas
Aquino
SARATOGA
ROAD
SARATOGA
244
BM
LAWRENCE
212
200
100
Campbell
350
300
905000
910
2000
0
2000
4000 Feet
N
LOS GATOS 2 MI.

Figure 4.4


ALMADEN ROAD
SELECTIVE MITIGATION PLANTINGS
EL RIO DRIVE
STEPPED GABIONS ON SLOPE
2% ±
EXIST. GROUND
PROPOSED CHANNEL WIDENING
EXISTING CHANNEL TO REMAIN

WIDENED GABION CHANNEL WITH BENCH



WIDENED CRIBWALL CHANNEL WITH MITIGATION BENCH

Figure 4.5

REACH 10A FEASIBLE ALTERNATIVES

TYPICAL SECTIONS

NTS

widened cribwall channel proposed for Reach 10A is shown in Figure 4.5). The cribwall is used to create a steeper side slope and allow for the establishment of the mitigation planting area.

V. SEDIMENT PROBLEMS

The District experiences sediment deposition in many of its flood protection facilities. The Multi-Year Stream Maintenance Program identified those areas most likely to require sediment removal activities as part of an ongoing program to maintain the function of the facilities (Figure 5.1). Approximately 100,000 cubic yards of sediments are removed annually. There are several known sources and causes creating the need for this sediment removal including tidal influence, historic facility design, and mining activities.

TIDAL INFLUENCE

Many of the District's early flood protection projects were in the low-lying areas which have daily tidal action. Fine sediment is carried by the tides into the south bay from the north bay through the Dumbarton narrows and is repeatedly suspended, circulated throughout the area, and deposited whenever and wherever the wave action subsides and the currents slow. Any region sheltered from winds and waves or strong currents that receives water by tidal flow from the mud flat areas will shoal rapidly (Krone, 1972). Generally, this reaches a climax elevation of about mean higher high water as shown in Figures 5.2A and 5.2B. Facilities which were designed and constructed without this information, like San Tomas Creek and the Sunnyvale channels, experience high maintenance costs to keep the facility operating properly. Calabazas Creek previously required the same intensive maintenance but the recent District project has incorporated the tidal impacts into the design (Calabazas). The new Calabazas levee height was established with the creek bottom flood bench at the tidal sediment elevation.

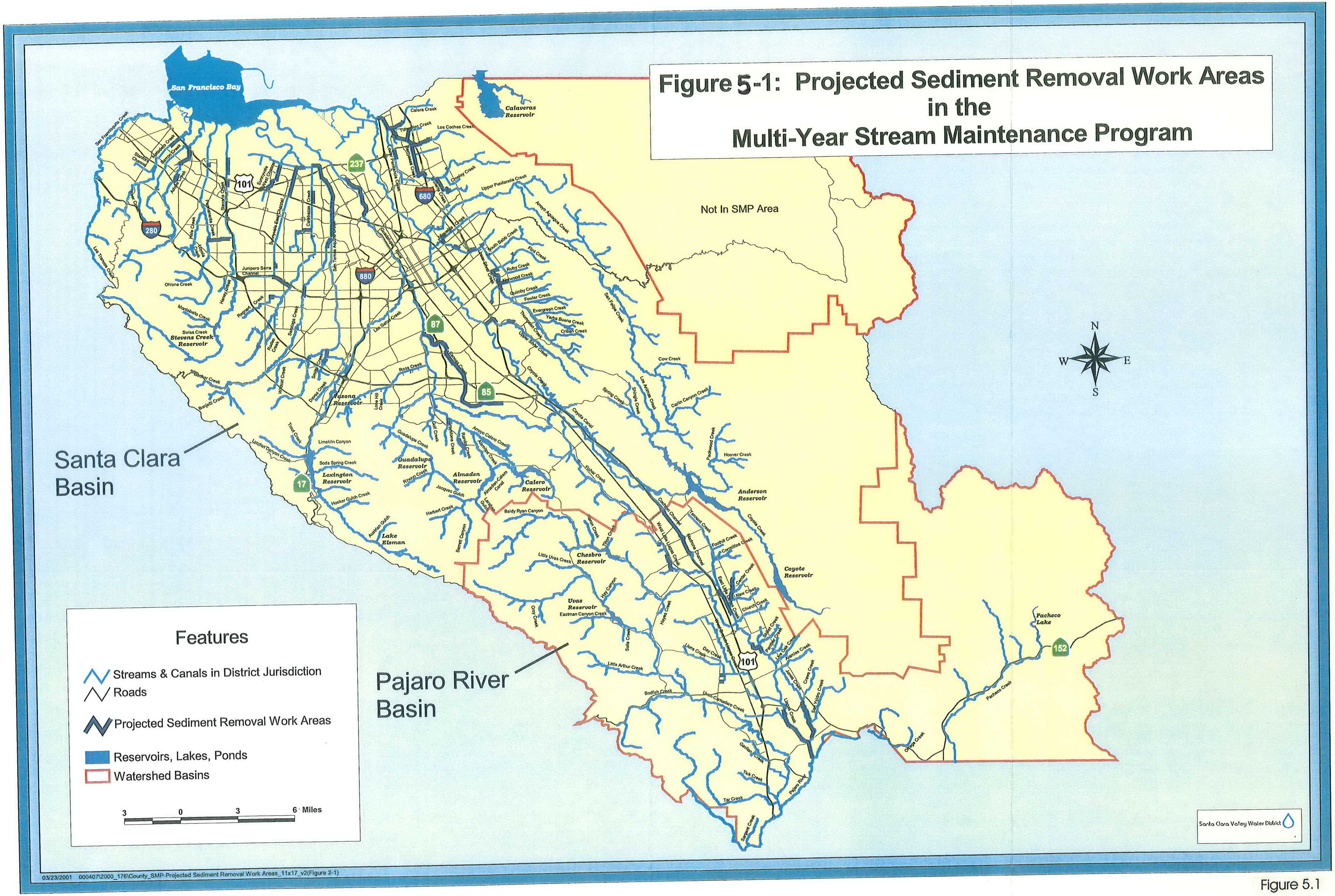
HISTORIC FACILITY DESIGN

Many flood protection facilities were designed and built without accommodating sediment or acknowledging the magnitude of the potential sediment removal requirements. Many channels such as San Tomas were designed with wide flat bottoms to be at a grade below mean high water. Many of these facilities were built with the understanding that the maintenance would be easy and not very costly. Other facilities, like Lower Silver Creek around Lake Cunningham, are the first phase of a bigger project and in the interim have become a sediment trap. Design of early District projects typically had 1½ to 1 side slopes and did not always acquire site-specific soil information to confirm the stability. Some of these creek banks continue to deteriorate a little every year because 1½ to 1 slope is too steep for the local soil stability.

MINING ACTIVITIES

At least two United States Geological Survey studies in Santa Clara County have identified quarrying operations as a major source of sediment in the watershed. The Calabazas Creek study found one tributary, with an abandoned, unreclaimed quarry, contributing 50 percent of the sediment in the watershed, a rate nearly nine times greater than other tributaries (Knott, 1978). The sediment yield on Permanente Creek was 3.5 times higher than expected under undisturbed conditions. One storm, estimated at a 10-year return period, produced 90 percent of all sediment transported during the 3-year study. The effective discharge transporting sediment in semi-arid and arid climates is a

Figure 5-1: Projected Sediment Removal Work Areas in the Multi-Year Stream Maintenance Program



03/23/2001 000407\2000_176\County_SMP-Projected Sediment Removal Work Areas_11x17_v2(Figure 2-1)

Figure 5.1

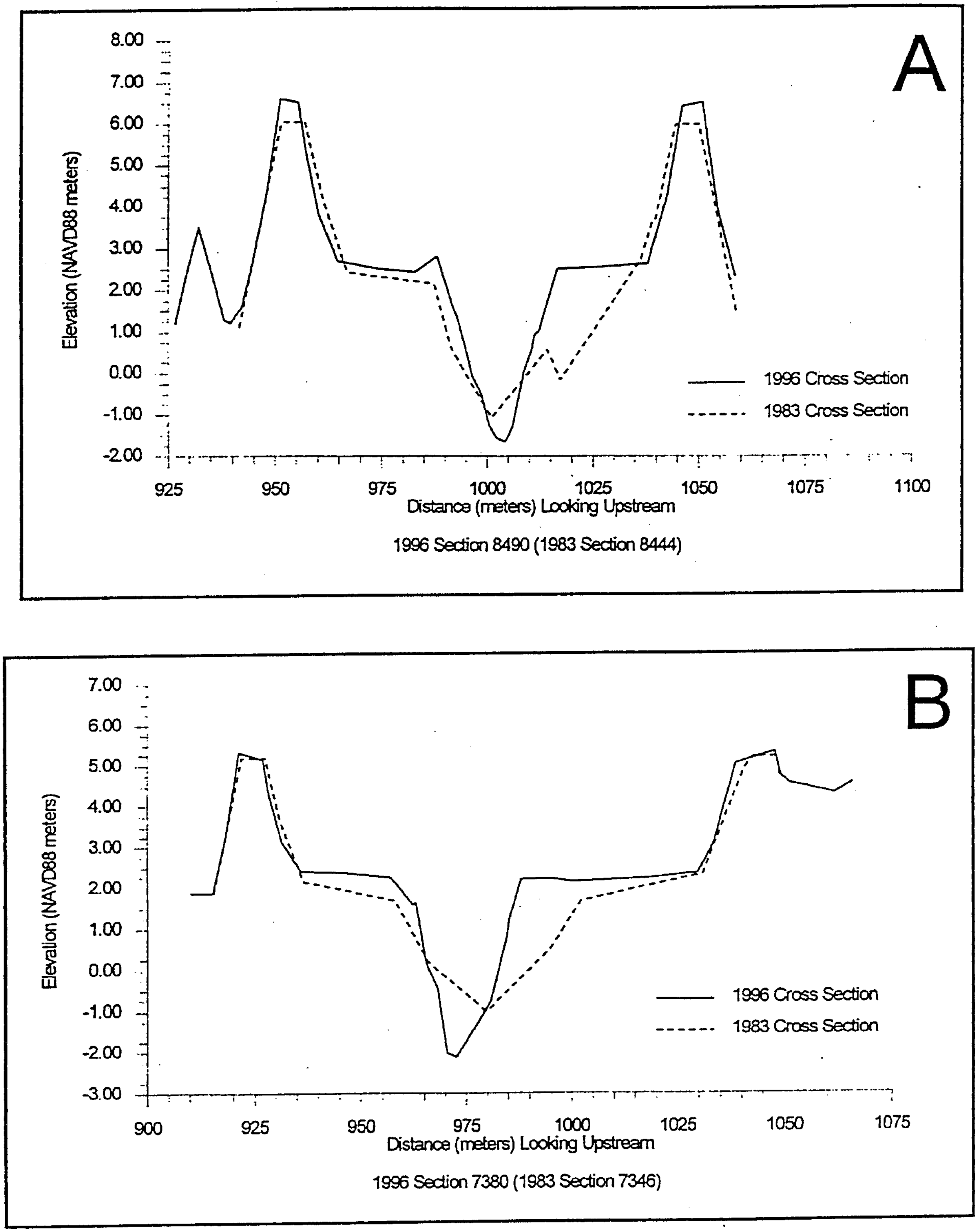


Figure 5.2 Cross-sections from 1983 and 1996 show changes in channel geometry at (A) located 460 m upstream of Hwy 237 and (B) located 270 m upstream of Gold Street bridge. The low flow channel exhibits varying degrees of incision and narrowing resulting from sediment accumulation between the leveed banks.

function of the more extreme events (Osterkamp, 2000, and Sloan, 2000). The increased sediment yield was apparently due to the availability of sediment from the quarry operation (Nolan, 1989).

There are numerous active and abandoned mines and quarries in Santa Clara County. The locations of the mines and the District's sediment removal areas are combined in Figure 5.3.

IMPACT OF UPPER GUADALUPE RIVER PROJECT

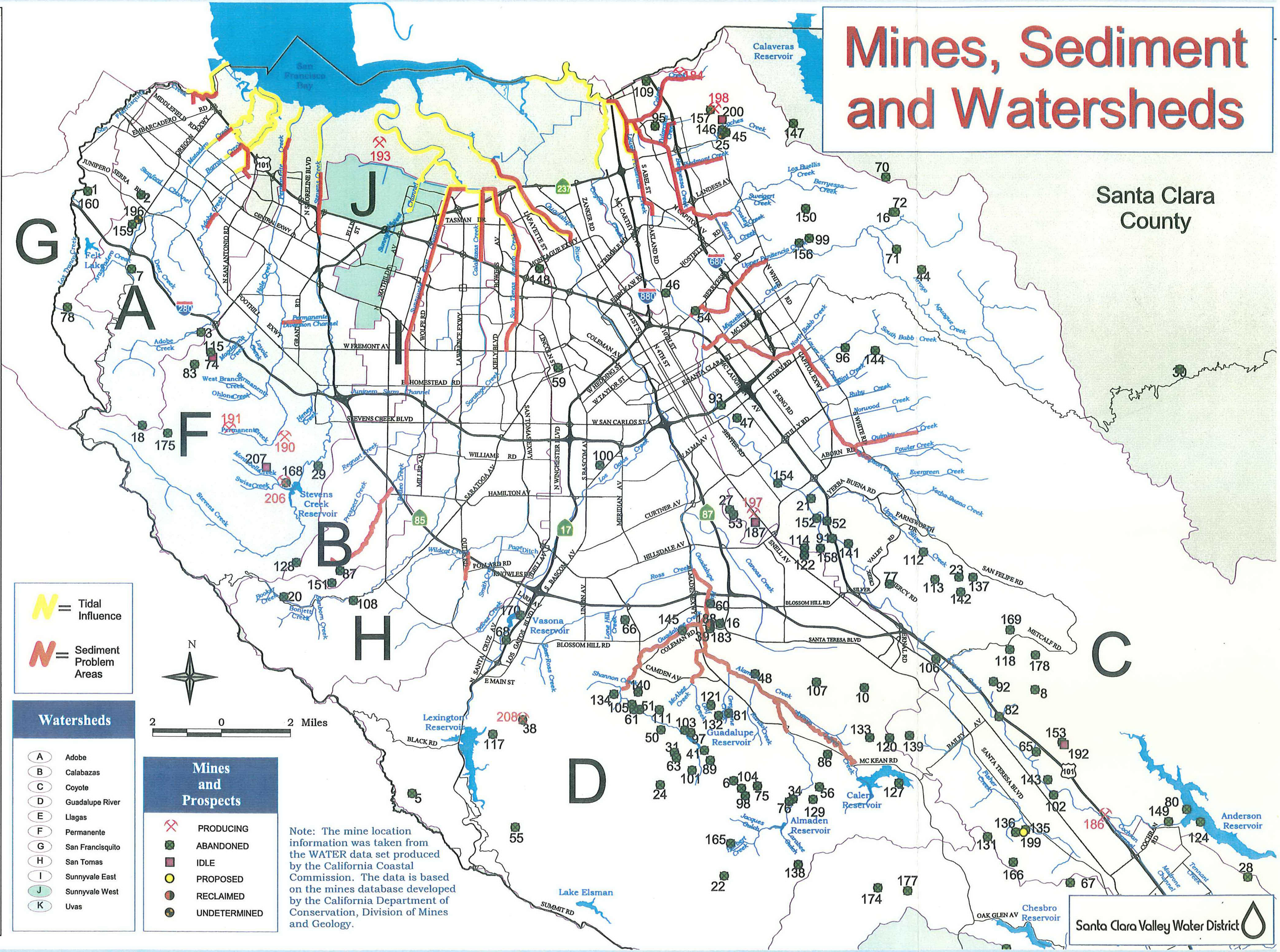
The Guadalupe River Watershed Planning Study identified only the Upper Guadalupe River reach between Branham Lane and Blossom Hill Road as an area of sediment deposition. This is a wide channel previously altered by quarry activity, where the flow velocity slows and deposits the sediment carried down from Guadalupe Creek.

The sediment transport capability was investigated to determine what, if any, adverse impacts would occur with the proposed channel improvements. The first study (Florsheim, 1993) recommended that the bypass channel be designed to function as a natural floodplain adjacent to the river and the elevation of the bypass channel be set at the channel forming discharge. Model results in a second analysis (Mead, 1996) concluded that the existing river had a relatively stable sediment transport regime and that the proposed flood protection measures would perform adequately relative to existing conditions.

Key	Mine Name	Mine Type	Status	Product
1	UNNAMED STONE QUARRY	UNKNOWN	ABANDONED	STONE
2	UNNAMED STONE QUARRY	SURFACE	ABANDONED	STONE
3	ADOBE CREEK QUARRY	SURFACE	ABANDONED	STONE
4	ALA MOUNTAIN MINE	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
5	ALMA	PROSPECT	ABANDONED	STONE
6	AMERICA MINE	UNDERGROUND	ABANDONED	MERCURY
7	BAHR & LEDOYEN, INC.	SURFACE	ABANDONED	STONE
8	BAY CITIES WATER COL	UNKNOWN	ABANDONED	CHROMITE
9	BEAUREGARD GROUP	UNDERGROUND	ABANDONED	MANGANESE
10	BERNAL MINE	SURFACE	ABANDONED	MERCURY
11	BILLY GOAT PROSPECT	UNKNOWN	ABANDONED	MANGANESE
12	BLACK BEAR MINE	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
13	BLACK EAGLE	SURFACE	ABANDONED	MANGANESE
14	BLACK HORSE CLAIM	SURFACE	ABANDONED	MANGANESE
15	BLACK WONDER (SEC.28) GROUP.	SURFACE	ABANDONED	MANGANESE
16	BLACKBIRD	PROSPECT	ABANDONED	MANGANESE
17	BLENDA	PLANT	ABANDONED	SAND AND GRAVEL
18	BOND	PROSPECT	ABANDONED	LIMESTONE
19	BONETTI	SURFACE	ABANDONED	CHROMITE
20	BORROW PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
21	BORROW PIT	SURFACE	ABANDONED	STONE
22	BOWIE	UNKNOWN	ABANDONED	MERCURY
23	BRADFORD RANCH	SURFACE	ABANDONED	MAGNESIUM
24	BRAINARD PROSPECT	UNDERGROUND	ABANDONED	MERCURY
25	CALAVERAS ROAD QUARRY	QUARRY	UNDETERMINED	STONE (BASE)
26	CEDAR TREE CLAIMS	SURFACE	ABANDONED	MANGANESE
27	CHABOYA	UNDERGROUND	ABANDONED	MERCURY
28	COCHRANE	PROSPECT	ABANDONED	MAGNESIUM
29	COMMERCIAL MATERIALS	SURFACE	ABANDONED	STONE
30	COPERNICUS PEAK PROSPECT	PROSPECT	ABANDONED	MANGANESE
31	COSTELLO MINE	PROSPECT	ABANDONED	MERCURY
32	DAVENPORT AND SMITH PROPERTY	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
33	DAVENPORT PROSPECT	PROSPECT	ABANDONED	MANGANESE
34	DEEP GULCH PLACER	UNKNOWN	ABANDONED	MERCURY
35	DEFOREST PROSPECT	UNKNOWN	ABANDONED	MANGANESE
36	DEPOSIT 8, BIG MATT	UNDERGROUND	ABANDONED	MANGANESE
37	DOAK NO 2 PROSPECT	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
38	DOUGLAS RANCH	SURFACE	ABANDONED	LIMESTONE
39	DOWNER PLANT	SURFACE	ABANDONED	STONE
40	ED KELLEY PROSPECT	SURFACE	ABANDONED	MANGANESE
41	ENRIQUITA MINE	UNDERGROUND	ABANDONED	MERCURY
42	FERRERO	SURFACE	ABANDONED	CHROMITE
43	FIDELITY DEPOSIT	SURFACE	ABANDONED	MAGNESIUM
44	FRANKEL PROSPECT	PROSPECT	ABANDONED	MANGANESE
45	FREDRICKSON-WETSON	SURFACE	ABANDONED	STONE
46	GARDEN CITY POTTERY	OPEN PIT	ABANDONED	CLAY (COMMON)
47	GLADDING BROS	OPEN PIT	ABANDONED	CLAY
48	GRAYSTONE QUARRY	PROSPECT	ABANDONED	STONE
49	GREAT EXPECTATION MINE	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
50	GUADALUPE LIME CO.	SURFACE	ABANDONED	LIMESTONE
51	GUADALUPE MINE	UNDERGROUND/OPEN PIT	ABANDONED	MERCURY
52	HELLYER PIT	UNKNOWN	ABANDONED	STONE
53	HILLSDALE	UNDERGROUND	ABANDONED	MERCURY
54	HILLSDALE PIT AND MILL	OPEN PIT	ABANDONED	SAND AND GRAVEL
55	HOOKER CREEK	UNDERGROUND	ABANDONED	COPPER
56	HOURET	PLANT	ABANDONED	LIMESTONE
57	HOWSON MILL	PLANT	ABANDONED	SAND AND GRAVEL
58	HOWSON PIT	UNKNOWN	ABANDONED	STONE
59	HUBBLE QUARRY AND MILL	UNKNOWN	ABANDONED	STONE
60	HUBBLE SAND AND GRAVEL PIT AND	OPEN PIT	ABANDONED	SAND AND GRAVEL
61	JOHN KNORR MINE	UNKNOWN	ABANDONED	STONE
62	KELLER BROTHERS PROSPECT	PROSPECT	ABANDONED	MANGANESE
63	KILDAY RANCH	SURFACE	ABANDONED	CHROMITE
64	LAUREL LAKE RANCH	UNDERGROUND/OPEN PIT	ABANDONED	CHROMITE
65	LEGUNA SCCA RANCH	PROSPECT	ABANDONED	COPPER
66	LONE HILL	SURFACE	ABANDONED	STONE
67	LOPEZ PROSPECT	UNKNOWN	ABANDONED	MANGANESE
68	LOS GATOS SAND & GRAVEL CO.	OPEN PIT	ABANDONED	SAND AND GRAVEL
69	MASSON RANCH	PROSPECT	ABANDONED	COPPER
70	MATEOS RANCH DEPOSIT	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
71	MCCOY	SURFACE	ABANDONED	MANGANESE
72	MILLER	SURFACE	ABANDONED	MANGANESE
73	MILLER AVENUE PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL (P.C.C.)
74	NEARY QUARRY	QUARRY	IDLE	STONE (BASE)
75	NEW ALMADEN MINE	UNDERGROUND/OPEN PIT	ABANDONED	MERCURY
76	NEW ALMADEN PLACERS & DUMPS	SURFACE	ABANDONED	MERCURY
77	NEW NORTH ALMADEN	SURFACE	ABANDONED	MERCURY
78	NEW PAGE MILL QUARRIES	SURFACE	ABANDONED	STONE
79	NEWSON PROSPECT	SURFACE	ABANDONED	MANGANESE
80	O'CONNELL BROTHERS	UNKNOWN	ABANDONED	MAGNESIUM
81	OAK HILL	SURFACE	ABANDONED	STONE
82	PACIFIC COAST AGGREGATES, INC.	OPEN PIT	ABANDONED	SAND AND GRAVEL
83	PATTON QUARRY	SURFACE	ABANDONED	STONE
84	PEBLE	SURFACE	ABANDONED	MANGANESE
85	PENNSYLVANIA	SURFACE	ABANDONED	MANGANESE
86	PERMANENTE METALS QUARRY	SURFACE	ABANDONED	STONE
87	PIERCE ROAD QUARRY	PROSPECT	ABANDONED	STONE
88	PINE RIDGE MINE	SURFACE	ABANDONED	MANGANESE
89	PROVIDENCIA MINE	UNDERGROUND	ABANDONED	MERCURY
90	PULSE PROSPECT	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
91	RAISCHE	OPEN PIT	ABANDONED	SAND AND GRAVEL
92	RANELLI	UNDERGROUND	ABANDONED	CHROMITE
93	REMILLARD-DANDINI CO.	OPEN PIT	ABANDONED	CLAY
94	RIANDA	UNDERGROUND	ABANDONED	MERCURY
95	ROGGASCH BROS.	PROSPECT	ABANDONED	STONE
96	ROGGASCH BROS. QUARRY	SURFACE	ABANDONED	STONE
97	SAN ANTONIO MINE	UNDERGROUND	ABANDONED	MERCURY
98	SAN FRANCISCO MINE	UNDERGROUND/OPEN PIT	ABANDONED	MERCURY
99	SAN JOSE	SURFACE	ABANDONED	MANGANESE
100	SAN JOSE BRICK & TILE	OPEN PIT	ABANDONED	CLAY (COMMON)
101	SAN JOSE CEMENT CO.	PROSPECT	ABANDONED	LIMESTONE
102	SAN JOSE PLANT	OPEN PIT	ABANDONED	SAND AND GRAVEL
103	SAN MATEO MINE	UNDERGROUND	ABANDONED	MERCURY
104	SAN PEDRO OPEN PIT	UNDERGROUND/OPEN PIT	ABANDONED	MERCURY
105	SANTA CLARA MILL	PLANT	ABANDONED	MERCURY
106	SANTA CLARA SAND & GRAVEL	OPEN PIT	ABANDONED	SAND AND GRAVEL
107	SANTA TERESA MINE	SURFACE	ABANDONED	MERCURY
108	SARATOGA	SURFACE	ABANDONED	STONE
109	SCOTT CREEK PIT	UNKNOWN	ABANDONED	STONE
110	SECURITY	SURFACE	ABANDONED	MAGNESIUM
111	SENATOR MINE	UNDERGROUND	ABANDONED	MERCURY
112	SILVER CREEK	UNDERGROUND/OPEN PIT	ABANDONED	MERCURY
113	SILVER CREEK HILLS PROSPECTS	SURFACE	ABANDONED	MERCURY
114	SMITH	SURFACE	ABANDONED	CHROMITE
115	SONDGRATH BROS.	SURFACE	ABANDONED	STONE
116	STANFORD QUARRY	SURFACE	ABANDONED	STONE
117	STEINHEIMER RANCH PROSPECT	PROSPECT	ABANDONED	CHROMITE
118	TILTON RANCH PROSPECT	PROSPECT	ABANDONED	MERCURY
119	TURNER DEPOSIT	UNKNOWN	ABANDONED	MANGANESE
120	UNNAMED CHROMITE PROSPECT	PROSPECT	ABANDONED	CHROMITE
121	UNNAMED CHROMITE PROSPECT	UNDERGROUND	ABANDONED	CHROMITE
122	UNNAMED CHROMITE PROSPECT	PROSPECT	ABANDONED	CHROMITE
123	UNNAMED CHROMITE PROSPECT	PROSPECT	ABANDONED	CHROMITE
124	UNNAMED GEMSTONE PROSPECT	PROSPECT	ABANDONED	GEMSTONES
125	UNNAMED GEMSTONE PROSPECT	PROSPECT	ABANDONED	GEMSTONES
126	UNNAMED GEMSTONE PROSPECT	PROSPECT	ABANDONED	GEMSTONES
127	UNNAMED LIMESTONE QUARRY	PROSPECT	ABANDONED	LIMESTONE
128	UNNAMED LIMESTONE QUARRY	SURFACE	ABANDONED	LIMESTONE
129	UNNAMED MANGANESE PROSPECT	PROSPECT	ABANDONED	MANGANESE
130	UNNAMED MERCURY PROSPECT	SURFACE	ABANDONED	MERCURY
131	UNNAMED MERCURY PROSPECT	PROSPECT	ABANDONED	MERCURY
132	UNNAMED MERCURY PROSPECT	UNDERGROUND	ABANDONED	MERCURY
133	UNNAMED MERCURY PROSPECT	UNDERGROUND	ABANDONED	MERCURY
134	UNNAMED MERCURY PROSPECT	SURFACE	ABANDONED	MERCURY
135	UNNAMED MERCURY PROSPECT	UNDERGROUND	ABANDONED	MERCURY
136	UNNAMED MERCURY PROSPECT	UNDERGROUND	ABANDONED	MERCURY
137	UNNAMED MERCURY PROSPECT	PROSPECT	ABANDONED	MERCURY
138	UNNAMED MERCURY PROSPECT	UNDERGROUND	ABANDONED	MERCURY
139	UNNAMED MERCURY PROSPECT	UNDERGROUND	ABANDONED	MERCURY
140	UNNAMED MERCURY PROSPECT	PROSPECT	ABANDONED	MERCURY
141	UNNAMED MERCURY PROSPECT	PROSPECT	ABANDONED	MERCURY
142	UNNAMED MERCURY PROSPECT	PROSPECT	ABANDONED	MERCURY
143	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
144	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
145	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
146	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
147	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
148	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
149	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
150	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
151	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
152	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
153	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
154	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
155	UNNAMED SAND AND GRAVEL PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
156	UNNAMED STONE QUARRY	PROSPECT	ABANDONED	STONE
157	UNNAMED STONE QUARRY	SURFACE	ABANDONED	STONE
158	UNNAMED STONE QUARRY	SURFACE	ABANDONED	STONE
159	UNNAMED STONE QUARRY	SURFACE	ABANDONED	STONE
160	UNNAMED STONE QUARRY	SURFACE	ABANDONED	STONE
161	UNNAMED STONE QUARRY	PROSPECT	ABANDONED	STONE
162	UNNAMED STONE QUARRY	PROSPECT	ABANDONED	STONE
163	UNNAMED STONE QUARRY	PROSPECT	ABANDONED	STONE
164	UNNAMED STONE QUARRY	PROSPECT	ABANDONED	STONE
165	UNNAMED STONE QUARRY	PROSPECT	ABANDONED	STONE
166	UNNAMED STONE QUARRY	PROSPECT	ABANDONED	STONE
167	UVAS PIT	OPEN PIT	ABANDONED	SAND AND GRAVEL
168	VOSS	SURFACE	ABANDONED	STONE
169	W.W. BURNETT RANCH	PROSPECT	ABANDONED	MAGNESIUM
170	WESTERN GRAVEL CO.	OPEN PIT	ABANDONED	SAND AND GRAVEL
171	WESTERN MAGNESITE COMPANY	SURFACE	ABANDONED	CHROMITE
172	WESTERN MINE	SURFACE	ABANDONED	MAGNESIUM
173	WESTERN MINE	SURFACE	ABANDONED	MANGANESE
174	WILLIAMS-RODRIQUES PROSPECT	UNDERGROUND/OPEN PIT	ABANDONED	MERCURY
175	WINSHIP	PROSPECT	ABANDONED	LIMESTONE
176	WINSHIP LEASES	UNDERGROUND/OPEN PIT	ABANDONED	MANGANESE
177	WRIGHT PROPERTY	SURFACE	ABANDONED	MERCURY
178	YETES LEASE	SURFACE	ABANDONED	CHROMITE
179	MEXICAN PROSPECT NO. 3	UNDERGROUND	IDLE	MANGANESE
180	ANTONIA CLAIM	OPEN PIT	IDLE	MANGANESE
181	KELLER PROSPECT	UNDERGROUND/OPEN PIT	IDLE	MANGANESE
182	QUEEN BEE PROSPECT NO.1	OPEN PIT	IDLE	MANGANESE
183	PIAZZA QUARRY	QUARRY	UNDETERMINED	STONE (BASE)
*184	CURTNER QUARRY	QUARRY	PRODUCING	STONE (BASE)
185	PACHECO CREEK PIT	OPEN PIT	RECLAIMED	SAND AND GRAVEL (FILL)
*186	POLAK PIT	OPEN PIT/QUARRY	PRODUCING	SAND AND GRAVEL (BASE)
187	HILLSDALE QUARRY	QUARRY	IDLE	STONE (BASE)
188	WINFIELD PIT	OPEN PIT	RECLAIMED	STONE (BASE)
189	ACME LODE PROSPECT	OPEN PIT	IDLE	MANGANESE
*190	PERMANENTE CEMENT PLANT	PLANT OR MILL	PRODUCING	PORTLAND CEMENT
*191	PERMANENTE QUARRY	QUARRY	PRODUCING	LIMESTONE (CEMENT)
192	O'CONNELL MINE	OPEN PIT	IDLE	CHROMITE
*193	BAY EVAPORATORS	EVAPORATION SYSTEM	PRODUCING	SALT
194	FABLE MINE	UNDERGROUND/OPEN PIT	IDLE	MANGANESE
195	SILVEIRA PIT	OPEN PIT	IDLE	SAND AND GRAVEL (BASE)
196	PAGE MILL QUARRY	QUARRY	UNDETERMINED	STONE (BASE)
*197	AZEVEDO QUARRY	QUARRY	PRODUCING	STONE (BASE)
*198	SERPA QUARRY	QUARRY	PRODUCING	STONE (SUBBASE)
199	TILTON RANCH QUARRY	QUARRY	PROPOSED	STONE (P.C.C.)
200	CALAVERAS QUARRY	QUARRY	IDLE	STONE (BASE)
201	BLACKBIRD CLAIM (BLACKHAWK)	OPEN PIT	IDLE	MANGANESE
202	COTTONWOOD CLAIM	OPEN PIT	IDLE	MANGANESE
203	JONES GROUP	UNDERGROUND/OPEN PIT	IDLE	MANGANESE
204	MAY BE CLAIM	OPEN PIT	IDLE	MANGANESE
205	MURMAC GROUP	OPEN PIT	IDLE	MANGANESE
*206	STEVENS CREEK QUARRY	QUARRY	PRODUCING	STONE (SUBBASE)
207	STEVENS CREEK QUARRY	QUARRY	IDLE	STONE (SUBBASE)
*208	LEXINGTON QUARRY	QUARRY	PRODUCING	STONE (BASE)

*NOTE: PRODUCING MINES

Mines, Sediment and Watersheds



Note: The mine location information was taken from the WATER data set produced by the California Coastal Commission. The data is based on the mines database developed by the California Department of Conservation, Division of Mines and Geology.

Figure 5.3

VI. PREVIOUS STUDIES INVESTIGATING THE FEASIBILITY OF FLOOD FLOW REDUCTION

The District has conducted numerous preliminary investigations of potential new storage facilities (Figure 6.1) and operational changes to existing facilities, which could reduce the flows in the Guadalupe River watershed. Some of these investigations have been single purpose objectives such as flood protection or water supply and others have been multipurpose including recreation.

NEW STORAGE FACILITIES

In 1972, the District prepared a summary (Church, 1972) of all earlier investigations. There was one investigation of enlarging Calero Reservoir and investigations of two sites in Guadalupe Creek.

The 1967 Calero Reservoir study was to establish the economic and engineering feasibility of enlarging Calero Dam and Reservoir to 24,000 acre-feet for water supply, recreation, and fish enhancement purposes. The project was dependent on state grant funding which did not become available. The expected flood protection benefits were considered to be very small. There would be some environmental impacts resulting from the inundation of tributary stream habitats.

In 1956, the Santa Clara Valley Water Conservation District investigated the engineering and economic feasibility of constructing a second dam and reservoir on Guadalupe Creek downstream of Shannon Road for the conservation of local runoff for groundwater recharge and flood protection. The reservoir was proposed to be 110 feet high with a storage capacity of 5,500 acre-feet. This is a new site and the construction of the dam and inundation of the main stem and tributaries would cause significant adverse impacts to several miles of mature riparian habitat and fishery resources including the endangered steelhead trout.

The "New Guadalupe Dam and Reservoir" study in 1968 to 1970 investigated a site on Guadalupe Creek on Hicks Road for a 24,300 acre-foot reservoir for water conservation, flood protection, and some recreation. A 3,000 acre-foot flood storage was reserved to limit the 100-year downstream flood flows on Guadalupe Creek to 1,600 cubic feet per second. No estimate was included on the hydrologic impacts downstream of the confluence with Alamitos Creek. This would be a new site and the construction of the dam and inundation of the main stem and tributaries would cause significant adverse impacts to several miles of mature riparian habitat and fishery resources.

Lexington Reservoir is a 19,834 acre-foot reservoir on Los Gatos Creek, which is a major tributary to the Guadalupe River. In 1974, the District conducted a preliminary economic feasibility study (Pandit, 1974) of enlarging the capacity to 40,600 acre-feet. Potential benefits included additional water supply, flood protection, and recreation. Expanded recreational use was found to be the critical factor in determining economic feasibility. Any flood protection benefits on the Guadalupe River would be limited to those reaches below St. John Street. There would be some environmental impacts resulting from the inundation of tributary stream habitats and from the relocation of the state highway.

In April 1977, the District studied (Talley, R., 1977) an off-stream storage alternative. This alternative consisted of expanding the existing Guadalupe River percolation ponds downstream of Blossom Hill Road to reduce the 1 percent flood peak. The entire site was to be excavated to gain a maximum flood flow reduction of 3,600 cfs. Channel modifications would still be necessary through most of Guadalupe River. This alternative was not feasible due to the high cost and the inadequate reduction of flow. This site has also been partially developed.

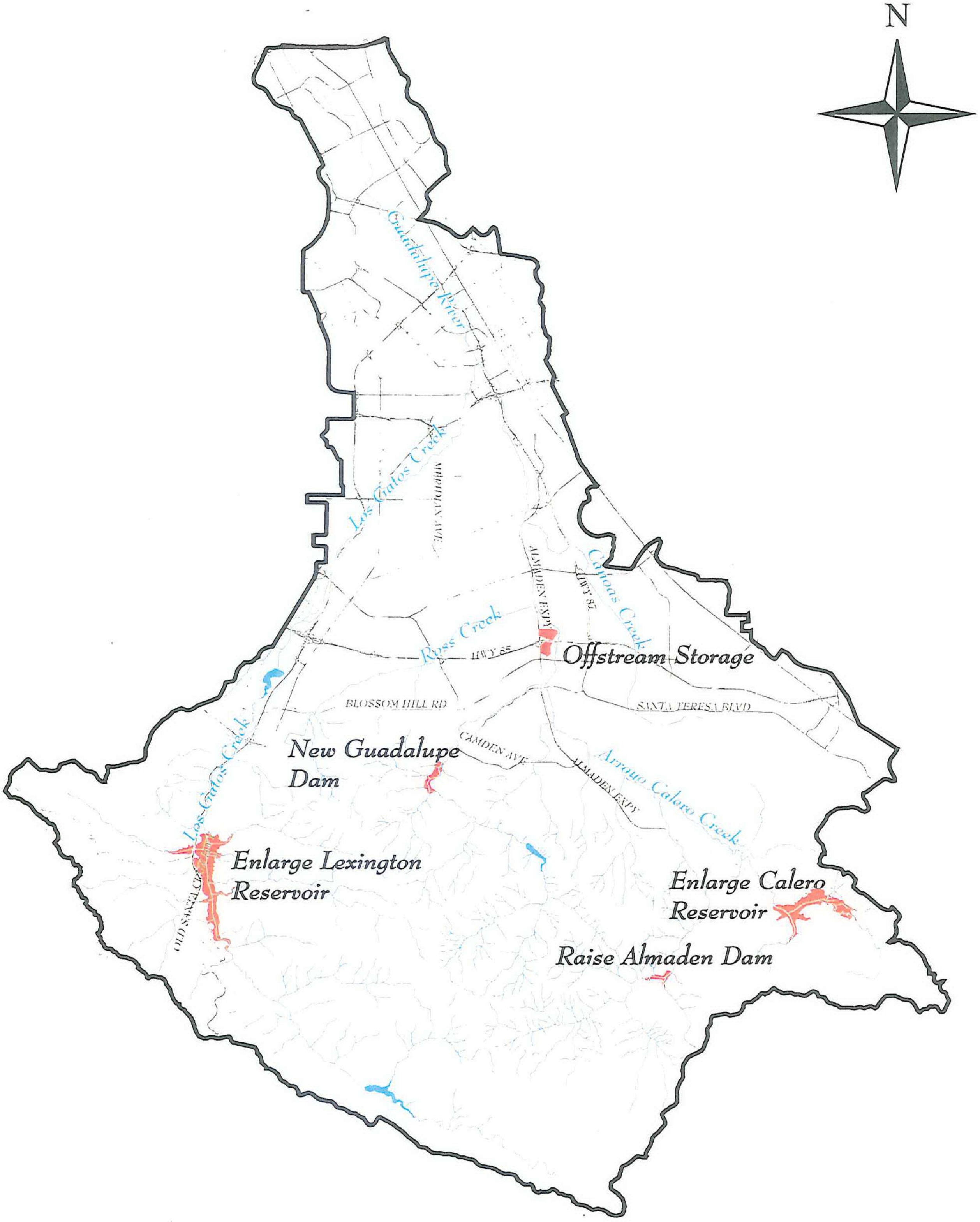


Figure 6.1 - INVESTIGATIONS OF POTENTIAL FLOOD FLOW REDUCTION

The Guadalupe River Watershed Planning Study Engineer's Report (Cheong, D., November 1999) revisited the feasibility of the earlier investigated sites and included additional investigations for enlarging Almaden Dam.

Text of these reports has been included in the appendices.

CONVERSION OF EXISTING RESERVOIRS

As part of the Guadalupe River Watershed Planning Study (Cheong, November 1999), the District investigated modifying the outlet works at Almaden, Calero, and Guadalupe Reservoirs and keeping them empty in the winter months to maximize flood capacity. The analysis concluded that the existing reservoirs do not have sufficient capacity or are not strategically located to have a significant impact on the downstream flood hydrograph. The 1 percent flow would only be reduced by 2,200 cfs which would still require extensive channel modifications in the upper Guadalupe River reaches. The acreage of impacts on riparian vegetation would still be significant. The District would have 13,900 acre-feet/year in local water supply yield. This change in reservoir function would have a substantial adverse effect in the availability of water and, therefore, the fisheries between May and September.

OPERATION OF EXISTING RESERVOIRS

Historically, the District's reservoirs were operated as single purpose with supply facilities. They were designed and operated to capture the winter runoff, eventually filling the reservoir and allowing any additional flows to pass over the spillway. There apparently was no consideration that the reservoirs would be operated as flood facilities requiring rapid evacuation of stored waters and therefore the outlet pipes were designed for water supply releases and not the much larger flood discharges. The reservoirs provide flood reduction benefits two ways: (1) The reservoir is likely to be less than full when a large storm occurs, and (2) Even a full reservoir will attenuate an inflow hydrograph because of the temporary surcharge storage created by flow over the spillway.

In recent years the District has developed "rule curves" for operating the reservoirs. The objective was to create additional flood protection benefits without sacrificing any water supply function.

The new operations were based on the probability that a reservoir could fill before the end of the rainy season. If a reservoir filled early such as November or December, the District could release some of the stored waters to create a small flood storage capacity through December or January and still expect that by April there would be sufficient additional runoff to fill the reservoir for water supply operations. The operating rule curves for Lexington and Almaden Reservoirs are shown in Tables 6.1 and 6.2. Use of the rule curves further attenuates the flood inflows beyond that which occurred with the historic single purpose operation.

Table 6.1

LEXINGTON RESERVOIR RULE CURVE

		DEC	JAN	FEB	MAR
1		14,000	14,000	15,104	16,997
2		14,000	14,000	15,175	17,056
3		14,000	14,000	15,247	17,114
4		14,000	14,000	15,318	17,172
5		14,000	14,000	15,389	17,230
6		14,000	14,000	15,461	17,288
7		14,000	14,000	15,532	17,346
8		14,000	14,000	15,604	17,404
9		14,000	14,000	15,675	17,462
10		14,000	14,000	15,747	17,520
11		14,000	14,000	15,818	17,578
12		14,000	14,000	15,889	17,636
13		14,000	14,000	15,961	17,694
14		14,000	14,000	16,032	17,752
15		14,000	14,000	16,104	17,810
16		14,000	14,065	16,168	17,909
17		14,000	14,129	16,232	18,008
18		14,000	14,194	16,297	18,107
19		14,000	14,258	16,361	18,206
20		14,000	14,323	16,425	18,305
21		14,000	14,387	16,489	18,404
22		14,000	14,452	16,554	18,503
23		14,000	14,516	16,618	18,602
24		14,000	14,581	16,682	18,701
25		14,000	14,645	16,747	18,800
26		14,000	14,710	16,811	18,899
27		14,000	14,774	16,875	18,998
28		14,000	14,839	16,939	19,097
29		14,000	14,903		19,195
30		14,000	14,968		19,294
31		14,000	15,032		19,393

*MINIMAL WATER SUPPLY IMPACT 12-Jan-98

Table 6.2

ALMADEN RESERVOIR FLOOD CONTROL CURVE

	DEC	JAN	FEB	MAR
1	800	882	1,005	1,130
2	800	887	1,009	1,135
3	800	892	1,012	1,139
4	800	897	1,016	1,144
5	800	902	1,019	1,149
6	800	906	1,023	1,154
7	800	911	1,027	1,159
8	800	916	1,030	1,164
9	800	921	1,034	1,168
10	800	926	1,037	1,173
11	800	931	1,041	1,178
12	800	935	1,044	1,183
13	800	940	1,048	1,188
14	800	945	1,052	1,193
15	800	950	1,055	1,197
16	805	953	1,061	1,216
17	810	956	1,066	1,235
18	815	960	1,071	1,253
19	819	963	1,077	1,272
20	824	966	1,082	1,290
21	829	969	1,087	1,309
22	834	973	1,093	1,327
23	839	976	1,098	1,346
24	844	979	1,103	1,364
25	848	982	1,109	1,383
26	853	985	1,114	1,402
27	858	989	1,119	1,420
28	863	992	1,125	1,439
29	868	995		1,457
30	873	998		1,476
31	877	1,002		1,494

VII. ALTERNATIVES FOR FURTHER REDUCTION OF FLOOD FLOWS

The District previously investigated reservoir options in the upper watersheds which could reduce flood flows (Chapter VI).

Additional flood storage facilities could be constructed in the urban areas which would further reduce flooding, erosion, and sedimentation problems by reducing runoff to the creeks. Since the reservoirs have substantially reduced flood flows and many of the creeks have been previously enlarged or realigned, it is not expected that further reduction in flows will significantly change the quasiequilibrium in the bankfull channel. Peak flow reduction could reduce the size and impacts of the proposed 1 percent flood protection measures.

IN-LINE STORAGE BASINS

One alternative investigated by the District for this study would be to store all the runoff volume from the urban area (3,064 AF) in large concrete holding basins beneath public roadways similar to the facilities in the City of San Francisco combined sewer system. For estimating purposes, the 18.5 square mile (11,900 acres) Canoas Creek watershed was used to represent the typical urban development pattern and density of all the urban area in the Guadalupe River watershed (approximately 2,000 parcels per square mile of urban developed area).

CANOAS WATERSHED CHARACTERISTICS

		Area			
	No. Parcels	Acres	Square Miles	% of Urban	% of Total Watershed
Urban Developed Area	____	7,850	12.3	100	66
Residential	24,600	4,440	6.9	57	37
Commercial and Industrial	330	970	1.5	12	8
Roads and Transportation	55	2,340	3.6	30	20
Other Urban	____	112	.2	1	1

Within the 12.3 square miles (7,850 acres) of urban developed area of the Canoas watershed, there are 702,000 linear feet of storm drain pipes contributing to 65 outfalls. Based on these values for a typical square mile of urban area, there are 5.3 storm drain systems consisting of 57,000 linear feet (10.8 miles) of pipe. The enlarged storage basins would be limited to those locations which already have a storm water collection system.

The reinforced concrete holding basins would be limited to 30-feet wide for construction purposes and placed beneath the roadways on the alignments of the existing storm drain network (Figure 7.1A). To maintain a positive gradient toward the creek and sufficient cover over the box, the height would be limited to 8 feet. This would create a storage capacity of 310 acre-feet (30x8x57,000/43,560) per square mile of urban area. In the Canoas watershed, this would provide 3,800 acre-feet (310x12.3 square miles of urban area) of storage capacity. This would be just sufficient to contain all of the Canoas creek runoff (3,700 AF) from the 1 percent flood. The cost of

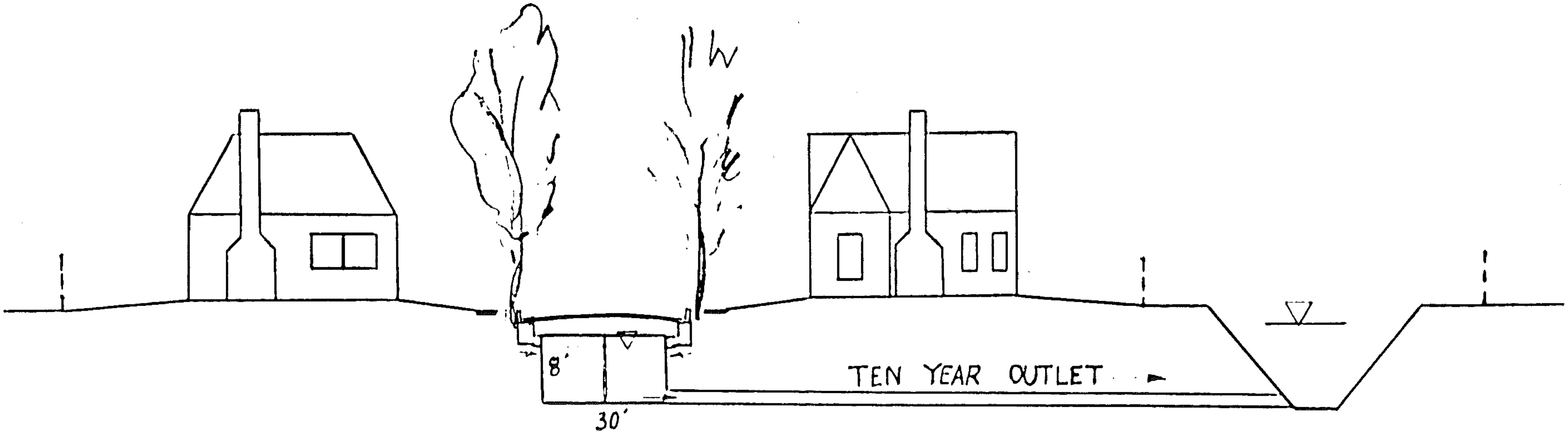


Figure 7.1 A

IN-LINE STORAGE BASINS

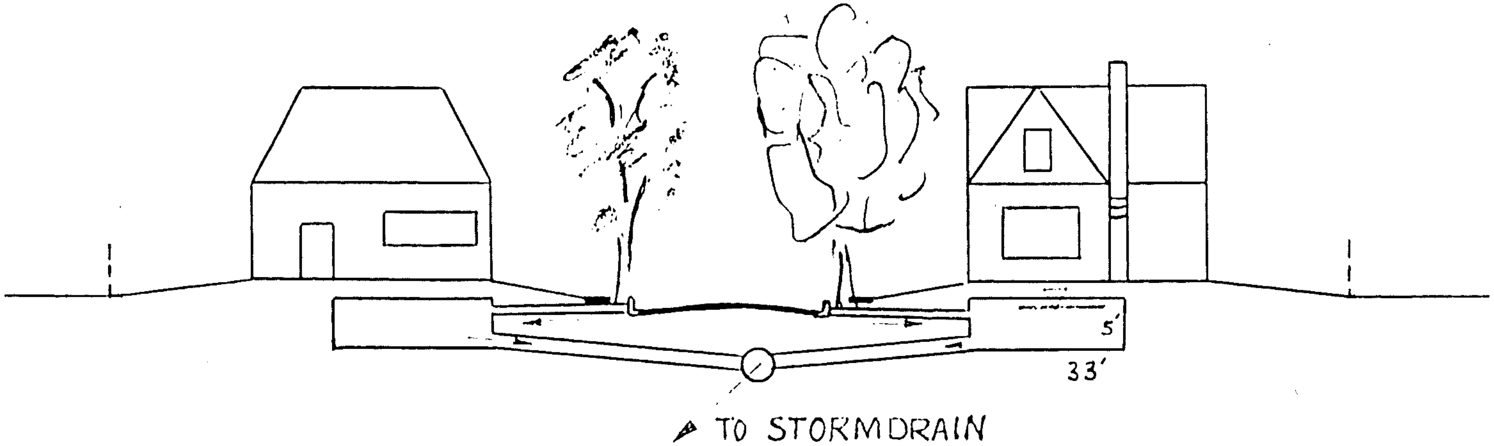


Figure 7.1 B

RETROFIT ON-SITE STORAGE

the in-line reinforced concrete box holding basins would be $2.9 billion (12.3 square miles urban area x $239,398,000 cost per square mile of urban area (see Appendix A).

If most flows are allowed to continue uninterrupted downstream to utilize available creek capacity, and only those creek flows in excess of the bankfull effective discharge are stored, then the volumes and costs drop significantly. Storing only flows in excess of a 10-year return period would reduce the storage requirements by nearly 60-70 percent. The cost of storing flood flows in excess of a 10-year event in an on-line holding basin system would be approximately $1.0-1.2 billion.

Based on the above estimates to provide a similar option to the 84.6 square miles of urban area in the entire Guadalupe River watershed would cost $8.3 billion (84.6/12.3x$1.2 billion).

The impact of collecting and storing all urban runoff, such that there is zero urban flow, with Guadalupe River 1 percent and 10 percent hydrographs, is shown in Figures 7.2, 7.3, 7.4, and 7.5. Reducing all the urban runoff to zero for every event would change the flood frequency distribution as shown in Figures 7.6 and 7.7. Allowing a 10-year urban flow to pass unimpeded to reduce the urban storage facility requirements would have less than half the impact reduction shown in Figures 7.6 and 7.7.

The critical feature in this, or any similar, system is the size of the storm drain pipe outfall to the creek. This pipe controls the peak flow rate that the urban area runoff will contribute to the creek flow. It also controls when and how much of the urban runoff is forced into temporary storage in the holding basins. If the outfall is designed for a 2-3 year return period, it substantially reduces the urban runoff flowing into the creek and substantially increases the amount of holding basin capacity needed. Increasing the outfall to a 10-year event would reduce the size and cost of the holding basin system. This same functional hydrologic relationship was a finding in the District's development of an urban hydrololgy methodology. An in-line storage basin system with a 10-year outfall would function similar to the existing urban development with the only difference being whether the urban runoff is temporarily stored in the streets or in concrete holding basins below the streets. The significant impacts of implementing this system are cost, traffic disruption during construction, and utility relocations.

RETROFIT ON-SITE STORAGE

Another option to create urban flood water storage would be to retrofit the 170,000 mostly residential parcels in the 84.6 square mile urban area of the Guadalupe River watershed with the equivalent of a 1,100 square foot basement about 5 feet high (Figure 7.1B). The basement holding basin would collect all the runoff from the individual parcel and adjacent roadway. Each holding basin would drain by gravity to a collection pipe in every street that would connect to the storm drain and eventually outfall to the creek. Using the Canoas Creek watershed development as representative of the entire Guadalupe River watershed urban area and the unit costs developed in Appendix A, the cost of retrofitting the existing structures to store all the urban runoff would be:

$150,000 per parcel x 170,000 parcels = $25.5 billion plus the additional 110,000 feet of pipe per square mile of urban area needed for the collection system:

$47,600,000 per square mile x 84.6 square mile = $4.0 billion.

Figure 7.2

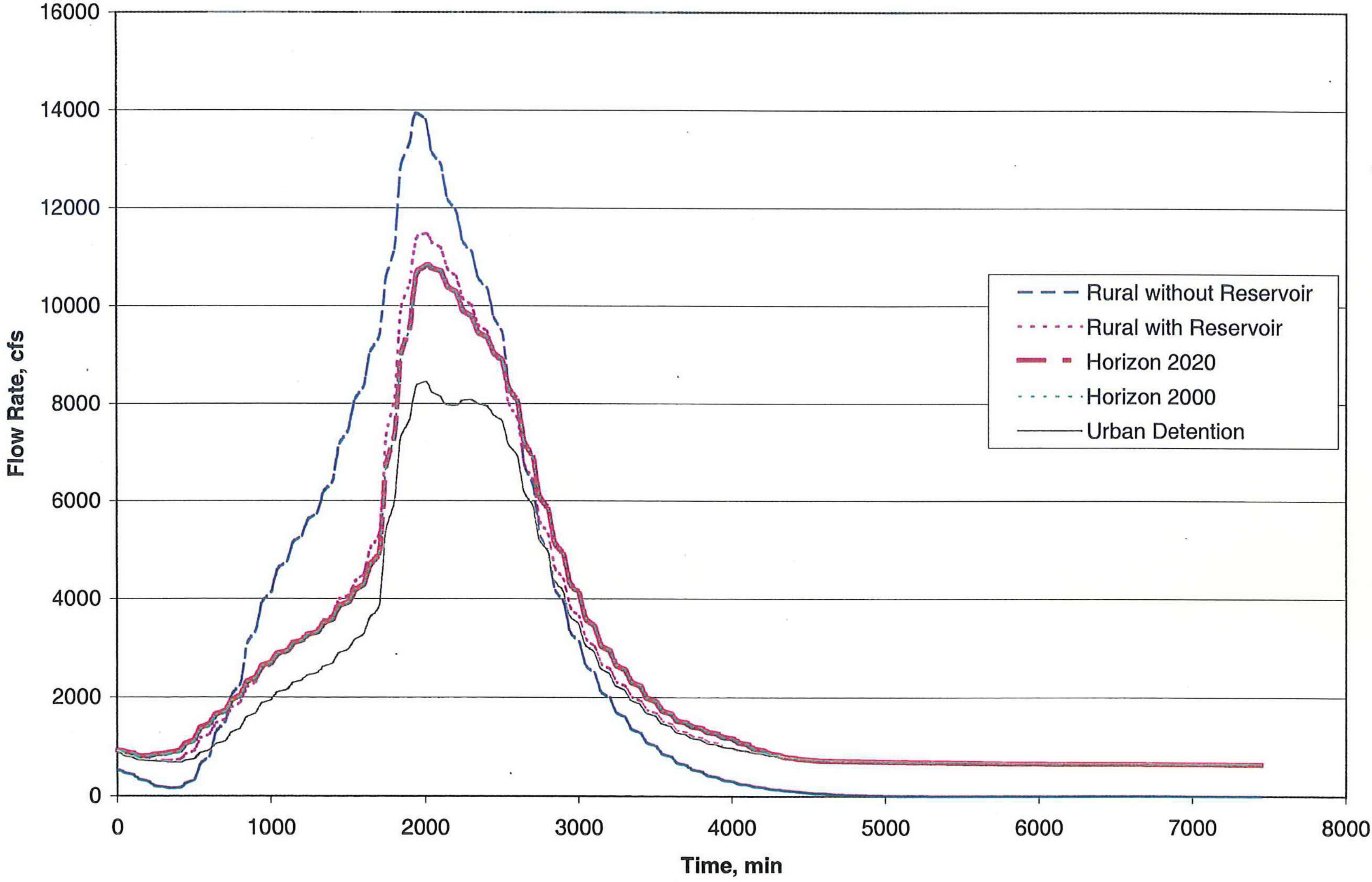

Guadalupe River D/S Alamitos Creek 100-Yr
Flow Rate, cfs
16000
14000
12000
10000
8000
6000
4000
2000
0
Time, min
0
1000
2000
3000
4000
5000
6000
7000
8000
Rural without Reservoir
Rural with Reservoir
Horizon 2020
Horizon 2000
Urban Detention

Figure 7.3

Guadalupe River D/S Los Gatos Creek 100-Yr

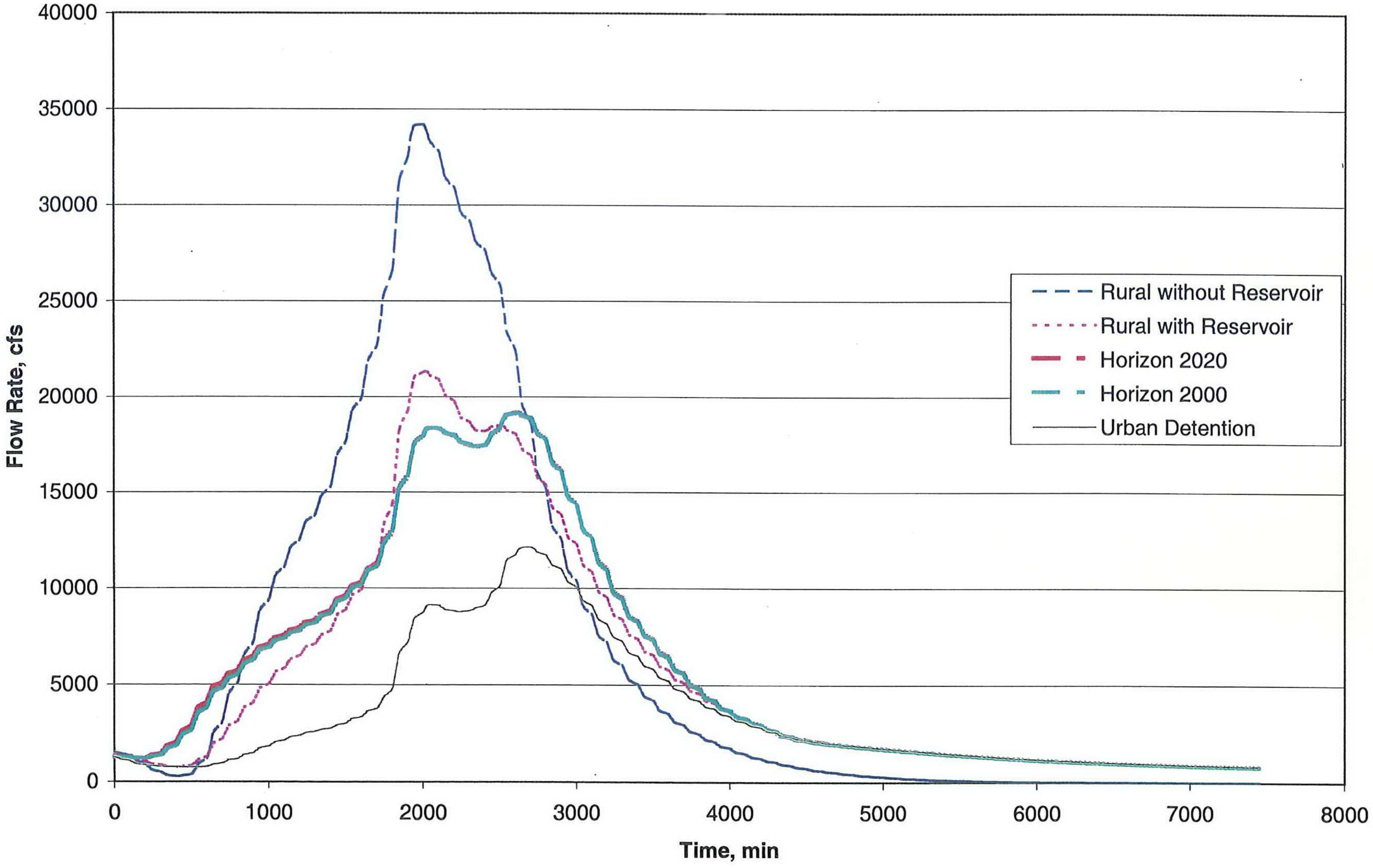


Figure 7.4

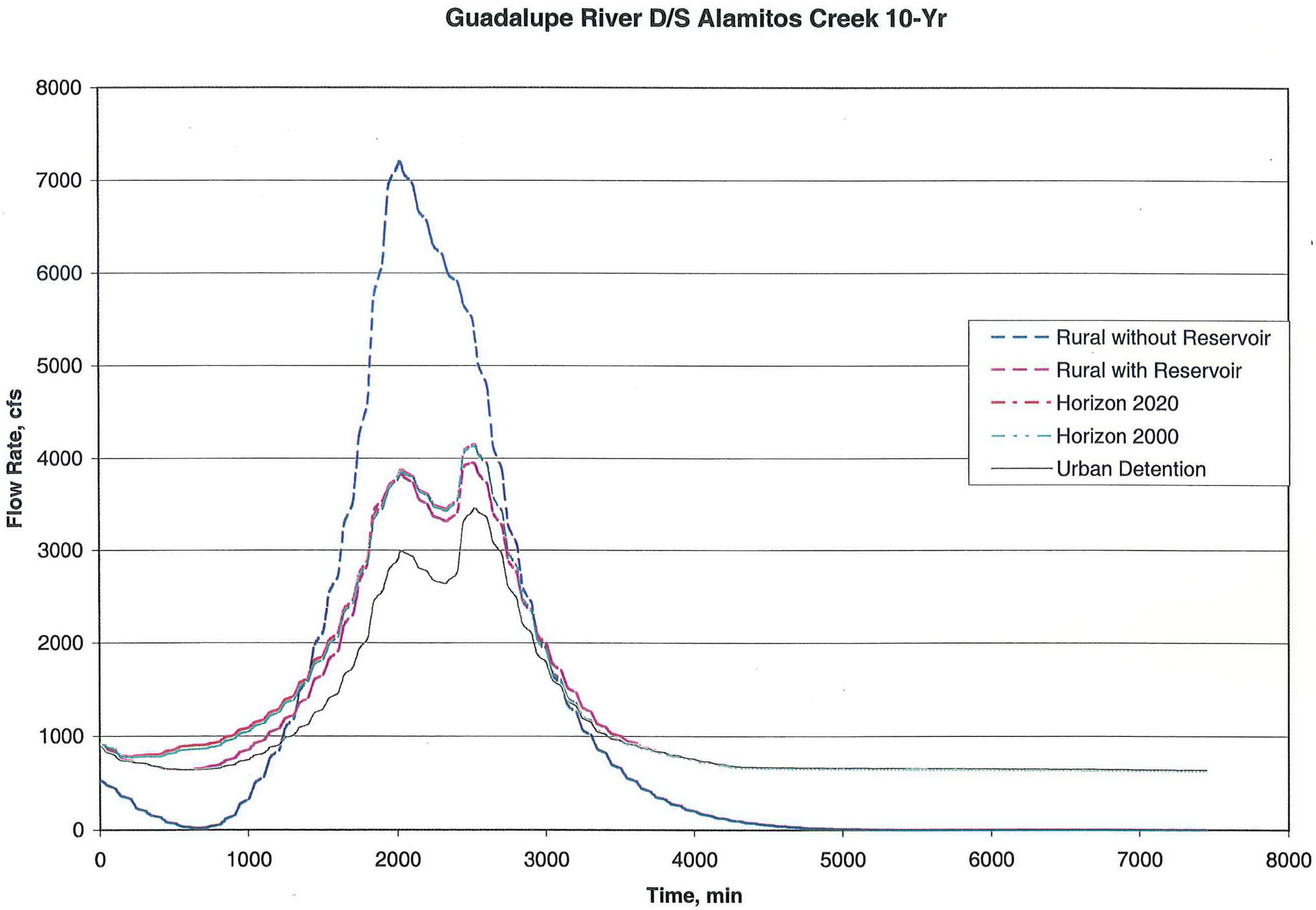

Guadalupe River D/S Alamitos Creek 10-Yr
Flow Rate, cfs
8000
7000
6000
5000
4000
3000
2000
1000
0
0
1000
2000
3000
4000
5000
6000
7000
8000
Time, min
Rural without Reservoir
Rural with Reservoir
Horizon 2020
Horizon 2000
Urban Detention

Figure 7.5

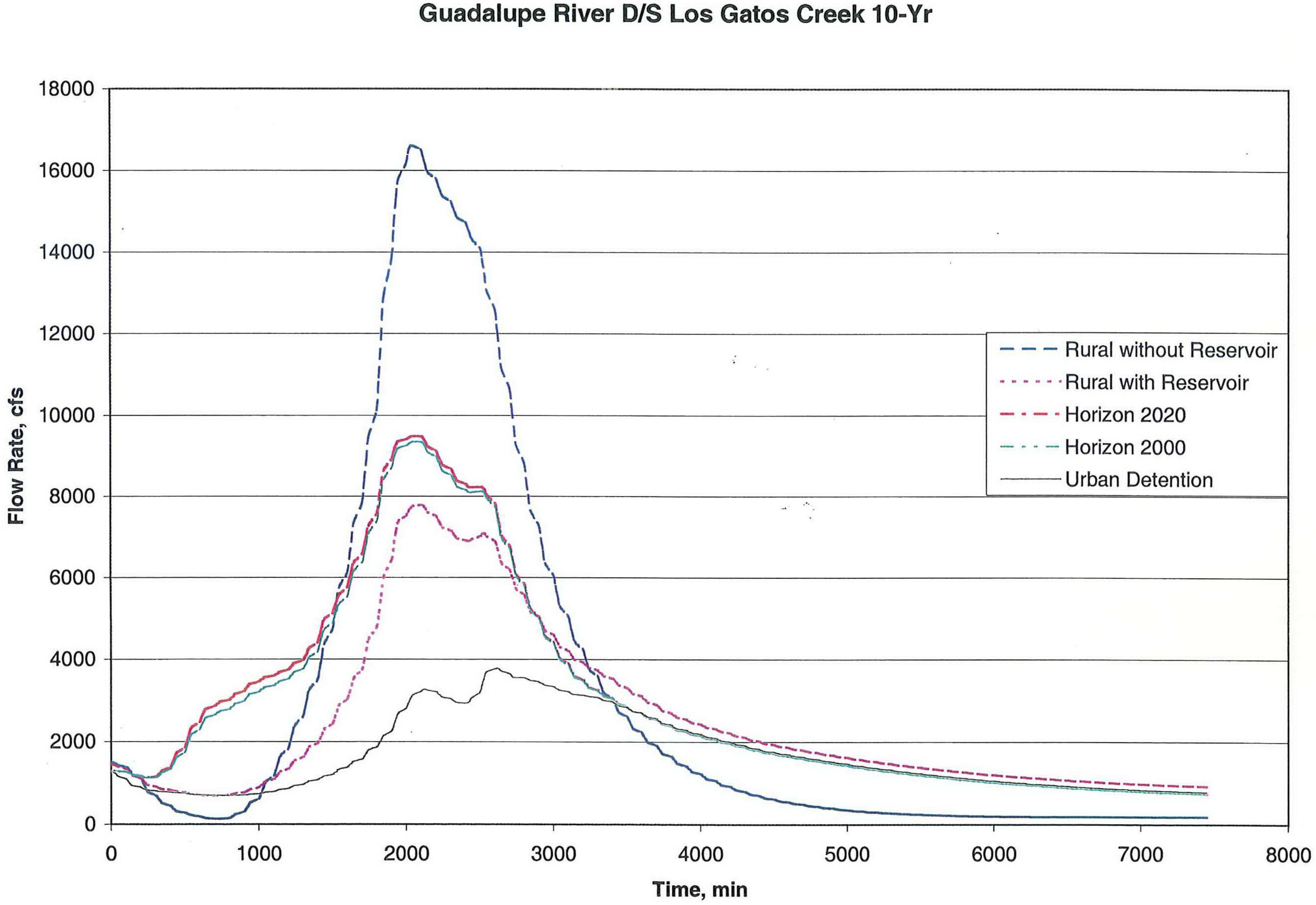
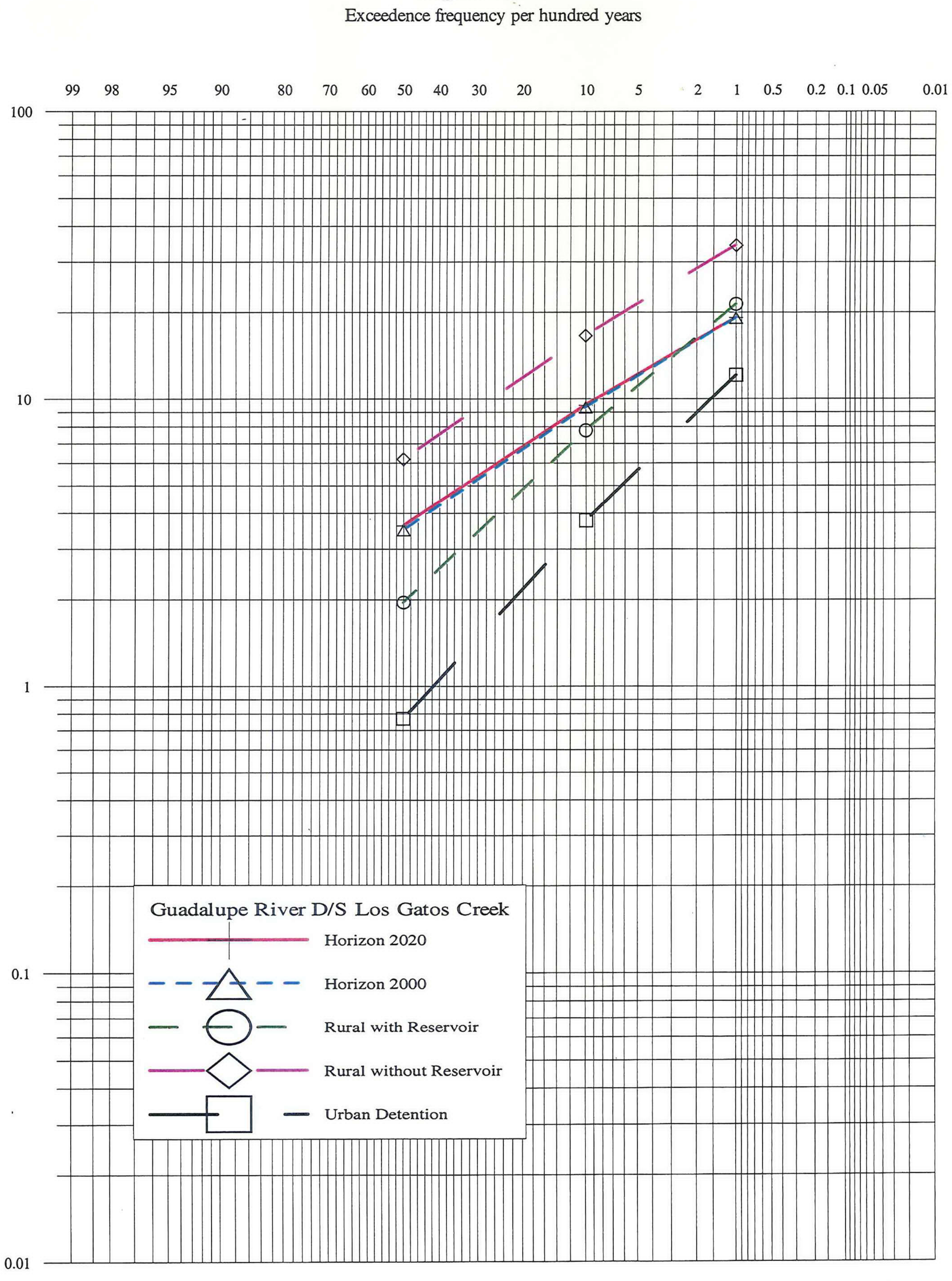
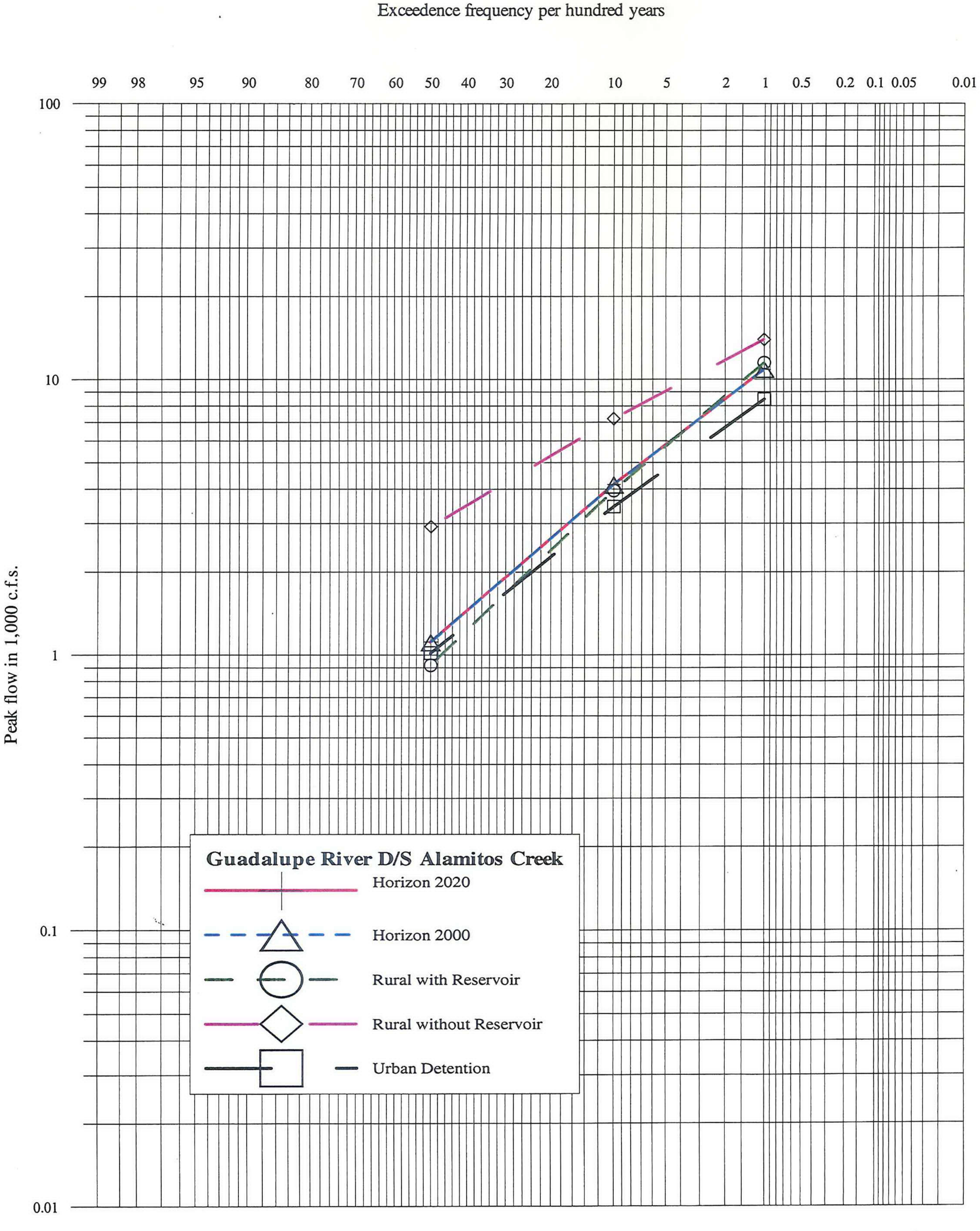

Guadalupe River D/S Los Gatos Creek 10-Yr
Flow Rate, cfs
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
1000
2000
3000
4000
5000
6000
7000
8000
Time, min
Rural without Reservoir
Rural with Reservoir
Horizon 2020
Horizon 2000
Urban Detention

Figure 7.6



Exceedence frequency per hundred years
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01
100
10
1
0.1
0.01
Peak Flow in 1,000 c.f.s.
Guadalupe River D/S Los Gatos Creek
Horizon 2020
Horizon 2000
Rural with Reservoir
Rural without Reservoir
Urban Detention

Figure 7.7



Exceedence frequency per hundred years
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01
100
10
1
0.1
0.01
Peak flow in 1,000 c.f.s.
Guadalupe River D/S Alamitos Creek
Horizon 2020
Horizon 2000
Rural with Reservoir
Rural without Reservoir
Urban Detention

The total cost for this alternative would be $29.5 billion.

This cost does **not** include acquisition of any rights of way. If the District constructed these facilities, a permanent right of way would be needed on each parcel.

This system would function similarly to the in-line holding basin alternatives. The sizing of the outfall storm drain will limit the peak flow from urban runoff entering the creek and will determine the amount of urban runoff that is forced into a temporary storage facility.

There would be significant challenges to implementing this system. The District or other public agencies could not spend public funds to build improvements on private property. Therefore, some permanent right of way would be needed at additional cost.

The approach would be for the collection system to be constructed by the District and the individual site improvements to be constructed by the property owners. The collection system would be designed similar to the in-line holding basins with sufficient capacity for containing the runoff from the roadways and public areas. The on-site improvements would be downsized to only handle the on-site runoff. Not all the system could function as a gravity system. Pumping would be required for some locations.

This system would provide the desired capacity and function but there is no effective mechanism for ensuring the improvements are constructed on the 170,000 parcels or ensuring that they will continue to be available in the future. This system would have significant impacts similar to the in-line holding basins. In addition, there would be significant social impacts resulting from the construction of the individual on-site holding basins.

VIII. IMPACT OF FLOOD FLOW REDUCTION ON UPPER GUADALUPE RIVER PROJECT

Providing additional capacity for storing floodwaters in the watershed either by upstream storage, off-stream storage, or urban storage would reduce the magnitude of the 1 percent flood used for design in the upper Guadalupe River project. This would potentially reduce the cross section, impacts and cost of the currently adopted measures and would depend on the amount of urban storage provided and flow reduction achieved.

The modifications to the proposed Upper Guadalupe River project would likely be:

REACHES 6-8

The adopted plan for flood protection improvements proposed to construct a secondary bypass channel parallel and adjacent to the existing river. A smaller 1 percent flood estimate would most likely result in some reduction in the width of the proposed bypass channel shown in Figures 8.1, 8.2, and 8.3.

REACHES 9 AND 10A

The adopted plan is for excavation of a high bench and cribwall construction on the east bank. With a lower 1 percent flood estimate, these measures could be reduced. The most likely change would be to raise the elevation of the bench. In some areas, the flood protection measures may be eliminated depending on how much the flow is reduced. See Figures 8.4 and 8.5.

REACH 10B

This reach is planned for restoration of the riparian habitat and geomorphological function. There would be no change if the design flow was reduced (Figure 8.6).

REACH 11

The east bank is proposed to be excavated and a cribwall constructed on the steep slope. A floodway, maintenance road, and revegetation strip would be located on the excavated bench. The most likely change with a reduced 1 percent design flow would be to either raise the bench, expand the revegetation strip or convert the steep cribwall to a flatter slope using gabions or similar material. See Figures 8.7 and 8.8.

REACH 12

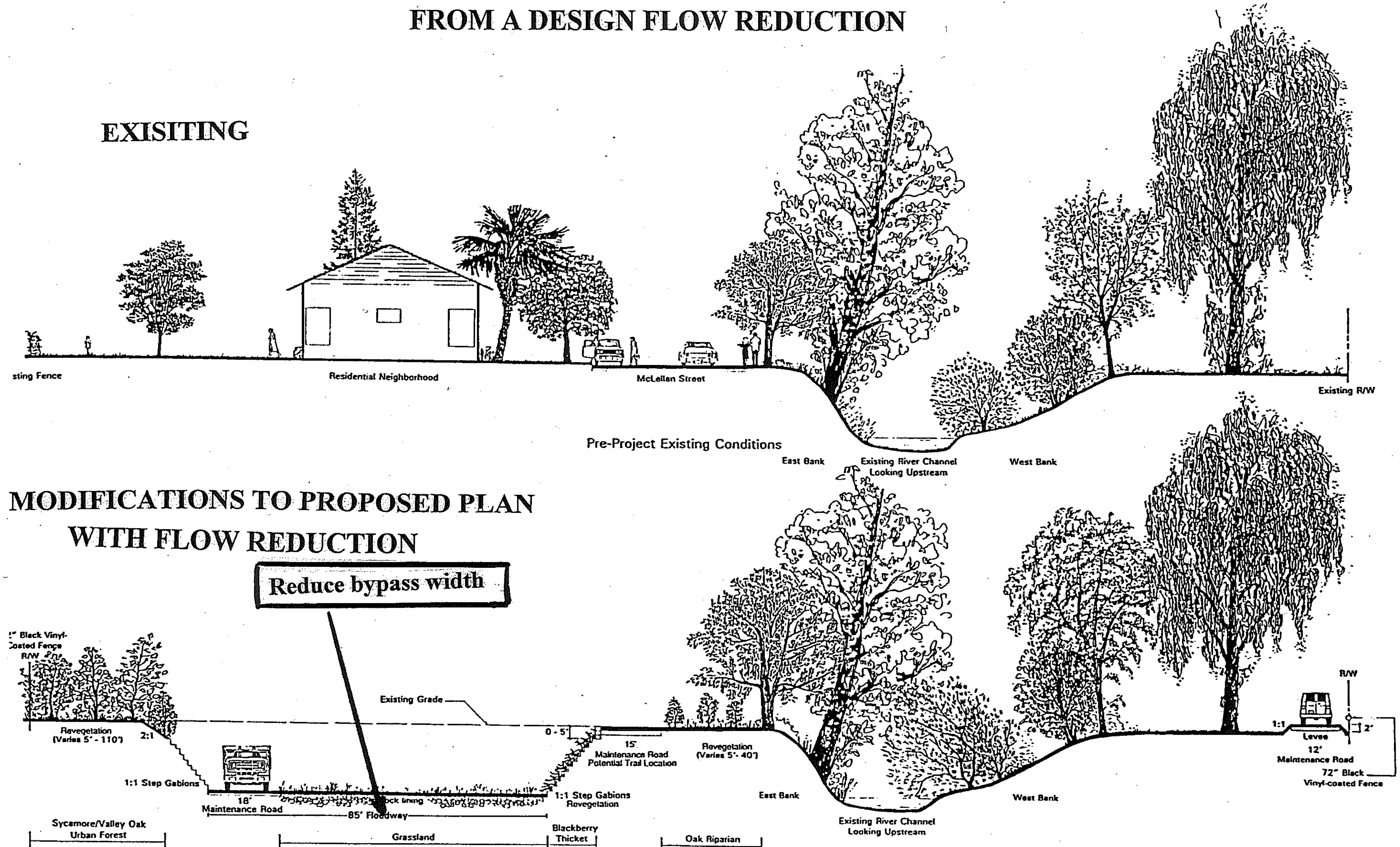
The proposed flood protection measures avoid the existing creek habitat, reconstruct the levees, and provide revegetation planting areas. There would be no change with a smaller design flow.

A summary of changes compared with the currently adopted plan are shown in Table 8.1.

Most of the channel modification costs were not changed for this analysis. There would be some reduction in cost if the design flows were reduced by a storage facility, but the right of way and major construction elements (gabions, cribwalls, etc.) would be basically the same as can be seen in Figures 8.1—8.6. The biggest difference would be the amount of material excavated to enlarge the channel capacity.

Figure 8.1

MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION



Pre-Project Existing Conditions

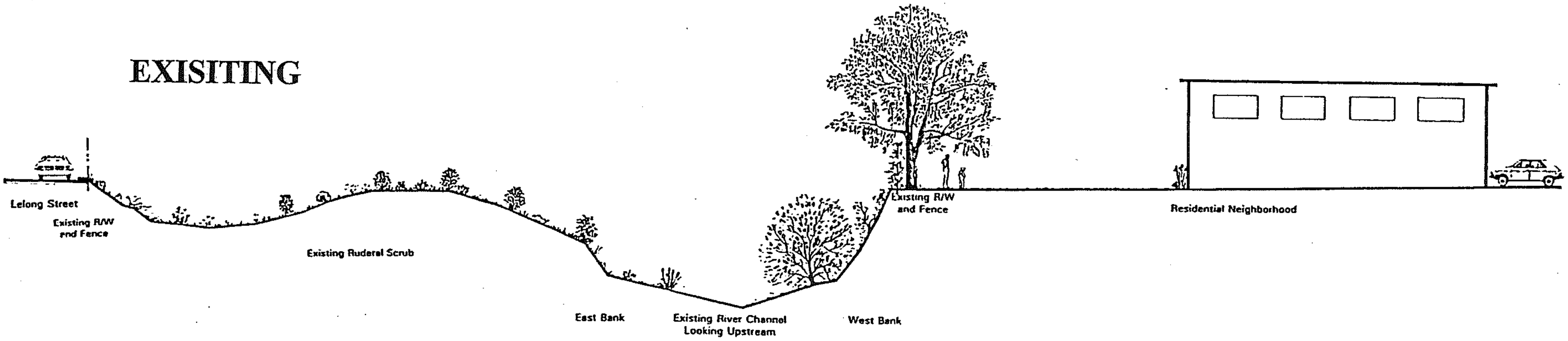
Post-Project 5 Years After Revegetation

Typical Section of Revegetation: 6 A - 6 A'
REACH 6: From Station 718 + 50 to Station 725 + 00

Figure 8.2

MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION

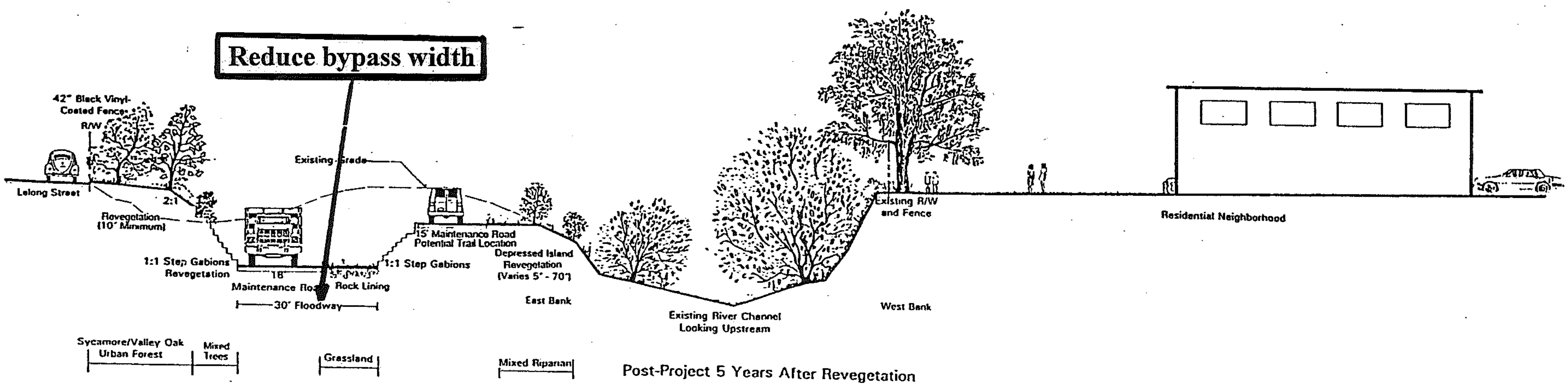
EXISITING



Pre-Project Existing Conditions

MODIFICATIONS TO PROPOSED PLAN WITH FLOW REDUCTION

Reduce bypass width


42" Black Vinyl-Coated Fence
R/W
Lelong Street
2:1
Revegetation (10' Minimum)
Existing Grade
1:1 Step Gabions Revegetation
18'
Maintenance Ro
5'
Rock Lining
30' Floodway
1:1 Step Gabions
15' Maintenance Road Potential Trail Location
Depressed Island Revegetation (Varies 5' - 70')
East Bank
Existing River Channel Looking Upstream
West Bank
Existing R/W and Fence
Residential Neighborhood
Sycamore/Valley Oak Urban Forest
Mixed Trees
Grassland
Mixed Riparian

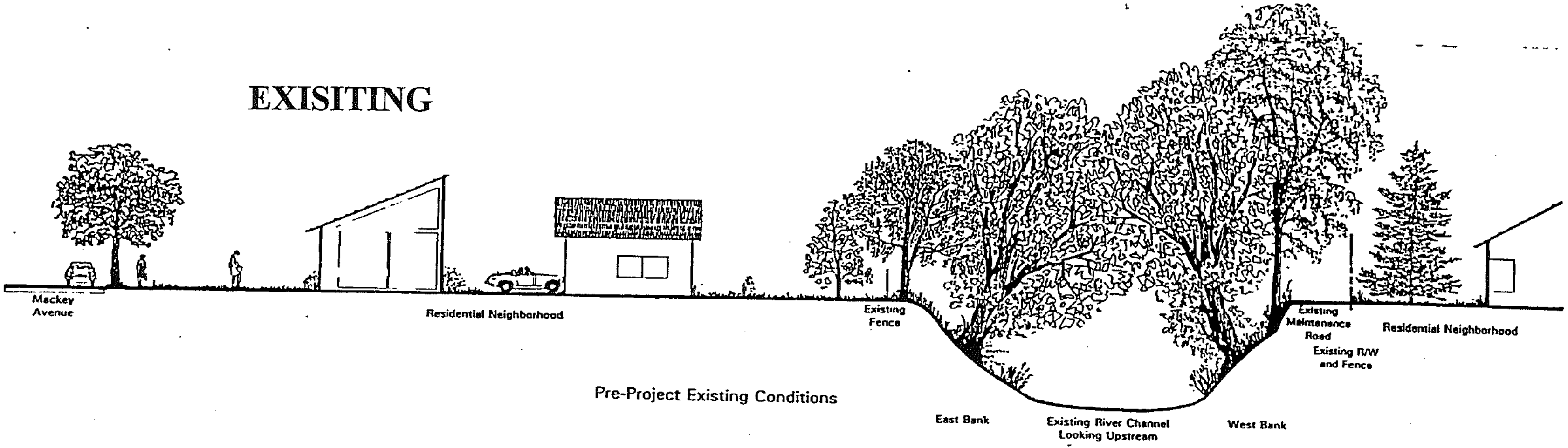
Post-Project 5 Years After Revegetation

Typical Section of Revegetation: 7 B - 7 B'
REACH 7: From Station 760 + 00 to Station 770 + 00

Figure 8.3

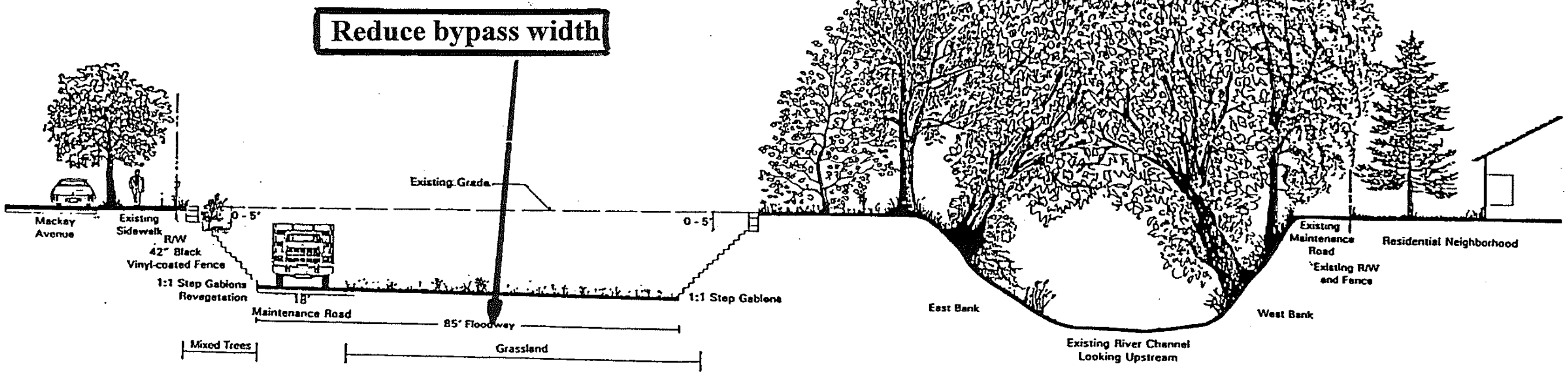
MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION

EXISITING



Pre-Project Existing Conditions

MODIFICATIONS TO PROPOSED PLAN WITH FLOW REDUCTION



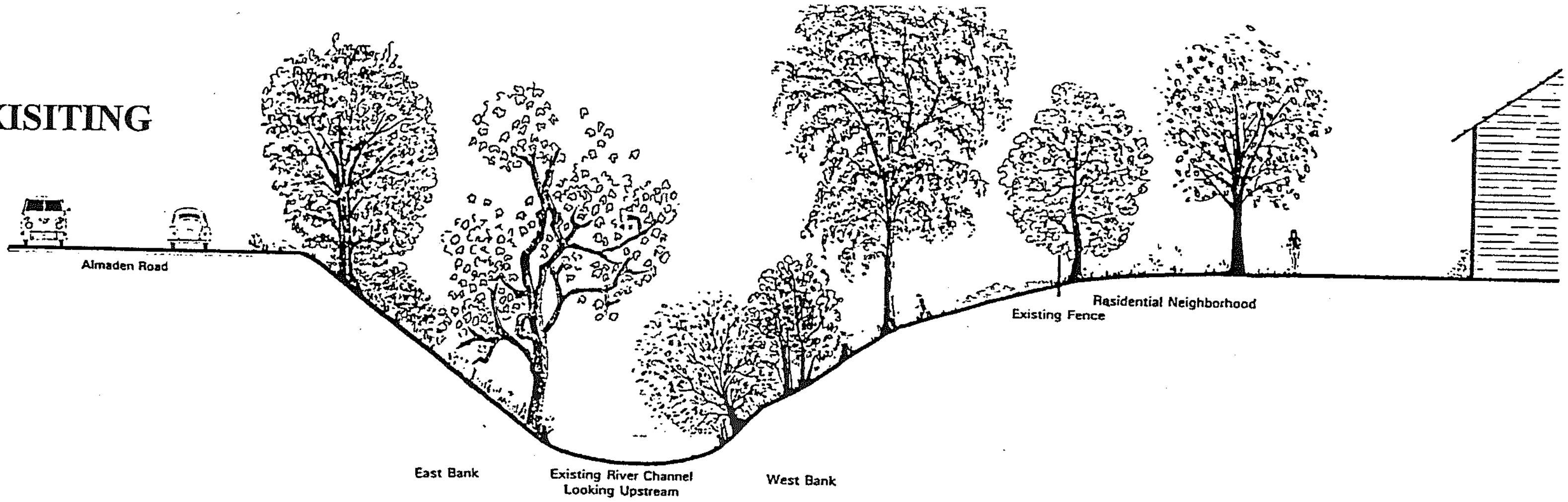
Post-Project 5 Years After Revegetation

Typical Section of Revegetation: 8 C - 8 C'
REACH 8: From Station 786 + 00 to Station 791 + 00

Figure 8.4

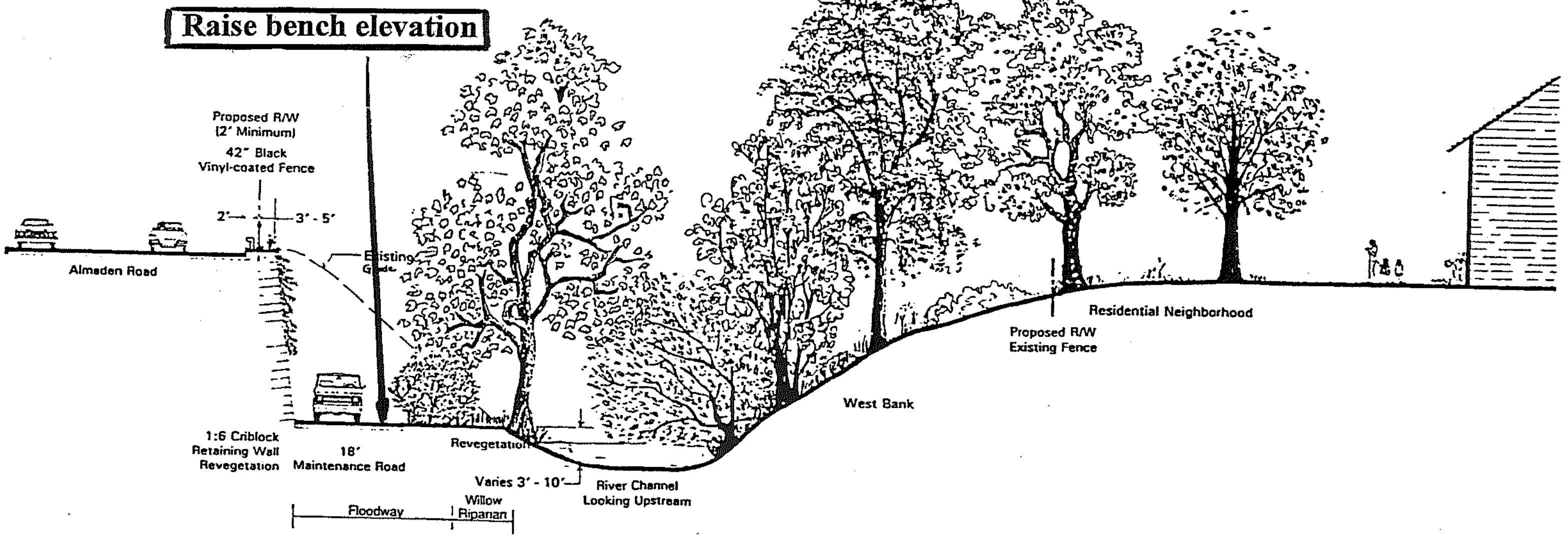
MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION

EXISITING



MODIFICATIONS TO PROPOSED PLAN WITH FLOW REDUCTION

Pre-Project Existing Conditions



Post-Project 5 Years After Revegetation

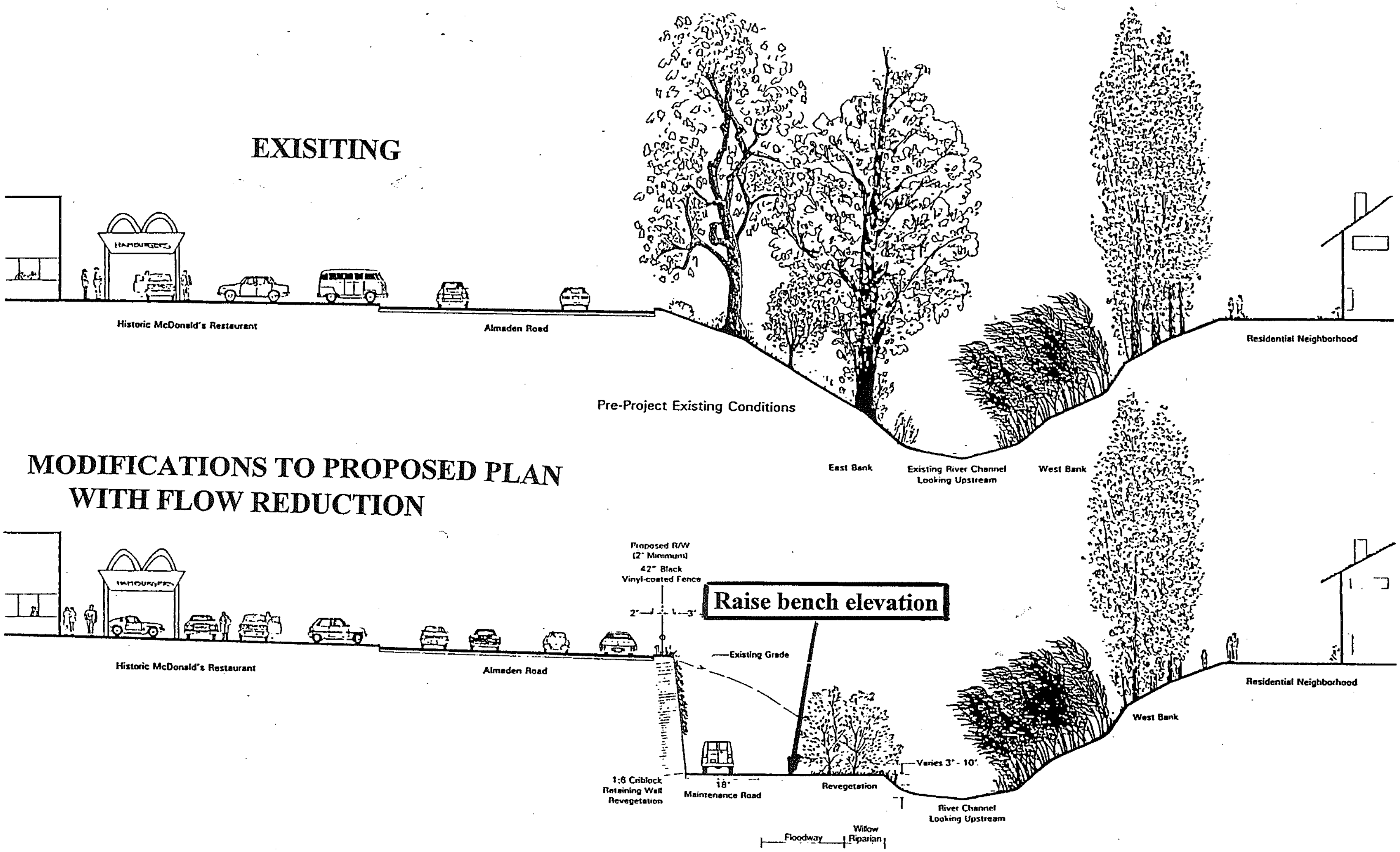
Typical Section of Revegetation: 9 D - 9 D'
REACH 9: From Station 831 + 00 to Station 842 + 00
East Bank Widening

Figure 3.5

MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION

EXISITING

MODIFICATIONS TO PROPOSED PLAN WITH FLOW REDUCTION


HAMBURGERS
Historic McDonald's Restaurant
Almaden Road
Pre-Project Existing Conditions
East Bank
Existing River Channel Looking Upstream
West Bank
Residential Neighborhood
Proposed R/W (2' Minimum)
42" Black Vinyl-coated Fence
2'
3'
Raise bench elevation
Existing Grade
1:6 Criblock Retaining Wall Revegetation
18'
Maintenance Road
Revegetation
Varies 3' - 10'
River Channel Looking Upstream
West Bank
Residential Neighborhood
Floodway
Willow Riparian

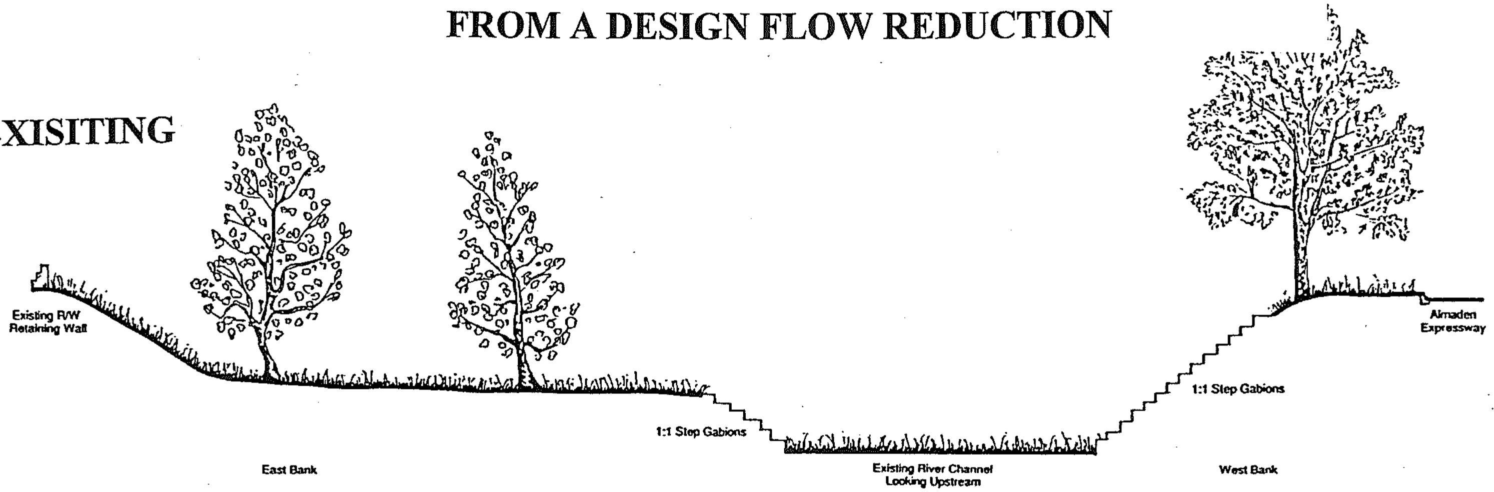
Post-Project 5 Years After Revegetation

Typical Section of Revegetation: 10A E - 10A E'
REACH 10A: From Station 844 + 00 to Station 851 + 00

Figure 8.6

MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION

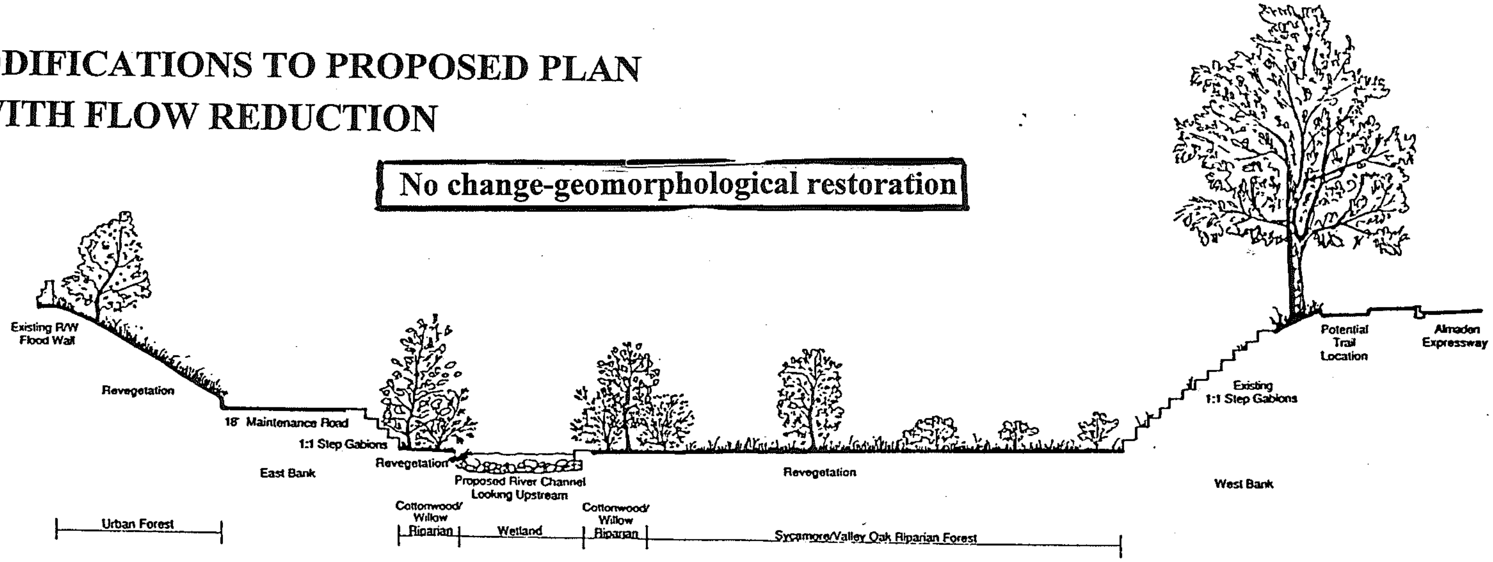
EXISITING



Pre-Project Existing Conditions

MODIFICATIONS TO PROPOSED PLAN WITH FLOW REDUCTION

No change-geomorphological restoration



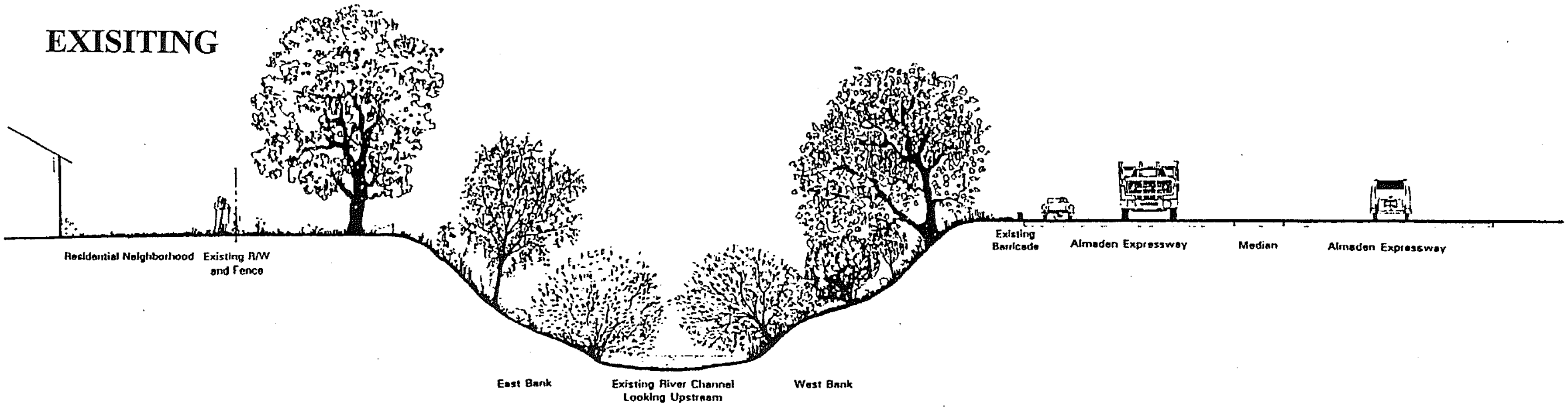
Post-Project 5 Years After Revegetation

Typical Section of Revegetation: 10B F - 10B F'
REACH 10B: From Station 876 + 00 to 877 + 00
Riparian Habitat Mitigation

Figure 8.7

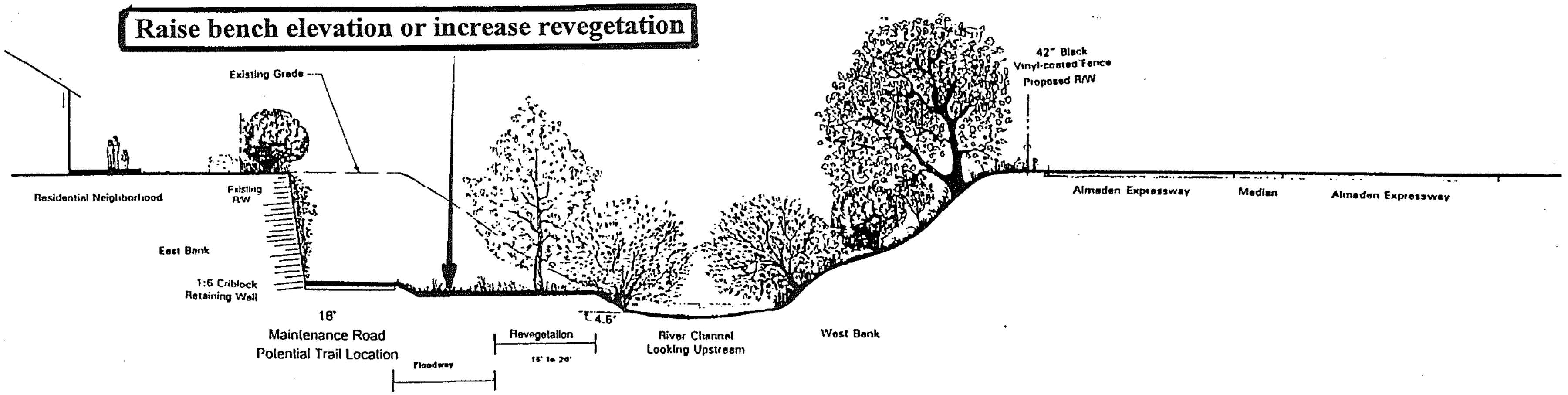
MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION

EXISITING



Pre-Project Existing Conditions

MODIFICATIONS TO PROPOSED PLAN WITH FLOW REDUCTION

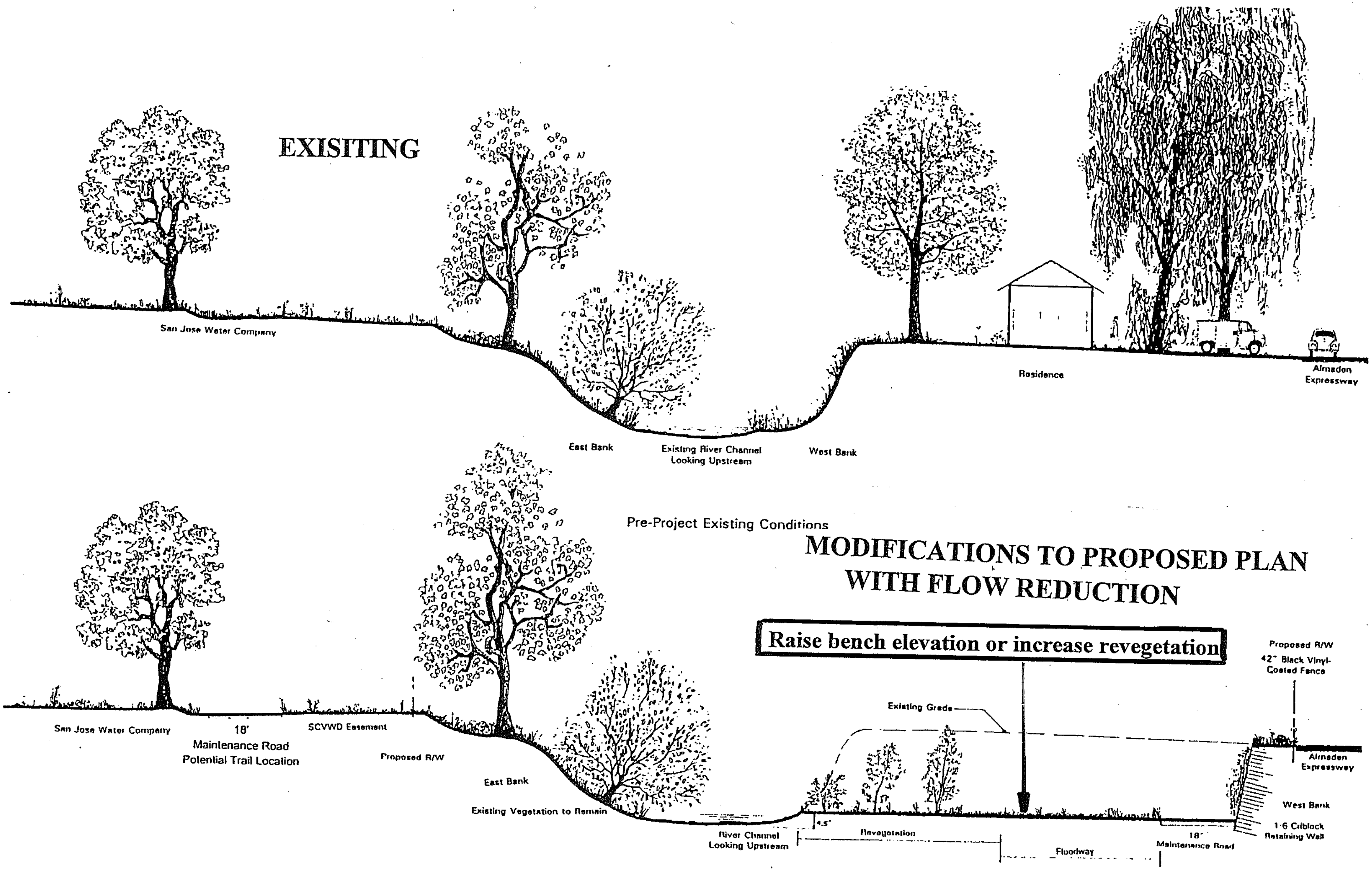


Post-Project 5 Years After Revegetation

Typical Section of R[illegible]tation: 11A H - 11A H
Reach 11A: From Sta[illegible]5+00 to Station 937+00
East Ba[illegible] Widening

Figure 8.9

MODIFICATIONS WHICH MOST LIKELY WOULD RESULT FROM A DESIGN FLOW REDUCTION



Post-Project 5 Years After Revegetation

Typical Section of Revegetation: 11B J - 11B J
Reach 11B: Fro[illegible]on 943+00 to Station 949+00
[illegible]ank Widening

TABLE 8.1
Summary Table for Guadalupe River Project Costs and Impact Acreage Reaches A, 6-12, Canoas Creek, and Ross Creek

Reach	Proposed Project	Upstream StoragePlan	Urban Storage
Reach A: Highway 101 to I-880	Levee/Floodwall	Somewhat lower Levee/Floodwall	Somewhat lower Levee/Floodwall
Construction Cost:	2.0 million	$2.0 million	$2.0 million
Total Cost:	$1.6 million		
Vegetation Impacted:	0 acres	0 acres	0 acres
Reach 6: I-280 to SPRR	Gabion Bypass Channel	Reduced width Gabion Bypass Channel	Reduced width Gabion Bypass Channel
Construction Cost:	$6.6 million	$6.6 million	$6.6 million
Total Cost:	$19.0 million	$19.0 million	$19.0 million
Vegetation Impacted:	0.3 acres	0.3 acres	0.3 acres
Reach 7: SPRR to UPRR	Gabion Bypass Channel	Reduced width Gabion Bypass Channels	Reduced width Gabion Bypass Channels
Construction Cost:	$12.1 million	$12.1 million	$12.1 million
Total Cost:	$21.0 million	$21.0 million	$21.0 million
Vegetation Impacted:	0 acres	0 acres	0 acres
Reach 8: UPRR to Willow Glen Way	Gabion Bypass Channel	Reduced width Gabion Bypass Channels	Reduced width Gabion Bypass Channels
Construction Cost:	$4.3 million	$4.3 million	$4.3 million
Total Cost:	$10.3 million	$10.3 million	$10.3 million
Vegetation Impacted:	0.3 acres	0.3 acres	0.3 acres
Reach 9: Willow Glen Way to Curtner Avenue	Widened Cribwall with Mitigation Bench	Widened Cribwall with higher Mitigation Bench	Widened Cribwall with higher Mitigation Bench
Construction Cost:	$10.0 million	$10.0 million	$10.0 million
Total Cost:	$13.8 million	$13.8 million	$13.8 million
Vegetation Impacted:	3.4 acres	Less than 3.4 acres	Less than 3.4 acres
Reach 10A: Curtner Avenue to Almaden Expwy (SB) and Canoas Creek	Widened Cribwall Channel with Mitigation Bench and 3.5 feet Floodwalls in Canoas Creek	Widened cribwall with higher bench Floodwalls in Canoas Creek	Widened cribwall with higher bench Floodwalls in Canoas Creek
Reach 10A:			
Construction Cost:	$2.9 million	$2.9 million	$2.9 million
Total Cost:	$3.5 million	$3.5 million	$3.5 million
Vegetation Impacted:	1.5 acres	Less than 1.5 acres	Less than 1.5 acres
Canoas Creek:			
Construction Cost:	$1.8 million	$1.8 million	$1.8 million
Total Cost:	$1.8 million	$1.8 million	$1.8 million
Vegetation Impacted:	0 acres	0 acres	0 acres
Reaches 10B & 10C: Almaden Expwy (SB) to Capitol Expwy	Restoration and Widened Gabion Channel with Bench u/s of Gage Sta. 23B	Restoration and Widened Gabion Channel with higher bench u/s of Gage Sta. 23B	Restoration and Widened Gabion Channel with higher bench u/s of Gage Sta. 23B
Construction Cost:	$6.4 million	$6.4 million	$6.4 million
Total Cost:	$9.7 million	$9.7 million	$9.7 million
Vegetation Impacted:	2.75 acres	Less than 2.75 acres	Less than 2.75 acres

TABLE 8.1
Summary Table for Guadalupe River Project Costs and Impact Acreage Reaches A, 6-12, Canoas Creek, and Ross Creek
(continued)

Reach	Proposed Project	Upstream StoragePlan	Urban Storage
Reach 11: Capitol Expwy to Branham Lane	East Bank Widened Gabion Channel With Bench	East Bank widened Gabion with higher bench	East Bank widened Gabion with higher bench
Construction Cost:	$2.5 million	$2.5 million	$2.5 million
Total Cost:	$10.6 million	$10.6 million	$10.6 million
Vegetation Impacted:	4.0 acres	Less than 4.0 acres	Less than 4.0 acres
Reach 12: Branham Lane to Blossom Hill Road	Levees construction and mitigation area	Levee construction and mitigation area	Levee construction and mitigation area
Construction Cost:	$4.1 million	$4.1 million	$4.1 million
Total Cost:	$14.4 million	$14.4 million	$14.4 million
Vegetation Impacted:	0.0 acres	0.0 acres	0.0 acres
Ross Creek	Widened Channel with articulated concrete mat lining	Widened Channel with articulated concrete mat lining	Widened Channel with articulated concrete mat lining
Construction Cost:	$3.9 million	$3.9 million	$3.9 million
Total Cost:	$3.9 million	$3.9 million	$3.9 million
Vegetation Impacted:	0.0 acres	0.0 acres	0.0 acres
New Storage Features		New Guadalupe Dam	In-Line holding basins
Construction Cost:	0		More than $1 billion
Total Cost:	0	More than $100 million	More than $1 billion
Vegetation Impacted:	0	Approximately 100 acres	0.0 acres
Guadalupe River, Alamitos Creek, and Guadalupe Creek	Fish Barrier Removal	Fish Barrier Removal	Fish Barrier Removal
Construction Cost:	$0.3 million	$0.3 million	$0.3 million
Total Cost:	$0.3 million	$0.3 million	$0.3 million
Vegetation Impacted:	0.0 acres	0.0 acres	0.0 acres
Total Project:			
Construction Cost:	$56.6 million	More than $200 million	More than $1 billion
Total Cost:	$109.7 million		More than $1 billion
Vegetation Impacted:	11.3 acres	More than 100 acres	Less than 11.3 acres

Note: Construction cost does not include mitigation cost

IX. RIPARIAN HABITAT MANAGEMENT AND LAND USE IN THE GUADALUPE RIVER WATERSHED

There are various ongoing integrated programs, practices, policies, and regulations affecting riparian habitat preservation and restoration in the Guadalupe River Watershed.

District

The District has different integrated water resources management programs that would improve the riparian habitat on the streams of the Santa Clara County (County), which includes Guadalupe River watershed. Some of those programs are:

- Flood Damage Reduction Program
- Environmental Programs
- Community Projects Review
- Water Supply Programs

Flood Damage Reduction Program

The District has a flood control program that systematically addresses flooding, erosion, sediment, maintenance and water quality issues in a multi-objective planning process. The District flood control projects undergo extensive planning and design before construction begins. Impacts of the projects on flows, fish and wildlife, vegetation, water quality, water supply, and homes and businesses within the watershed are all taken into account during the planning process. Public concerns are evaluated in a lengthy review process. The District also seeks to integrate other projects and programs in the watershed into the flood control projects.

The current schedule for implementation of projects in the Guadalupe River Watershed is:

Guadalupe River—Alviso to Interstate 880	2002
Guadalupe River—Highway 101 to Interstate 880 and Highway 280 to Blossom Hill Road	2002
Guadalupe River—Highway 880 to Highway 280	2002

Environmental Programs

District staff prepare environmental documents, evaluate natural resources, and develop and implement mitigation programs. Great emphasis is focused on environmental protection, including the reestablishment and maintenance of streamside habitats. Some of the programs include:

- Vegetation Management Program: This program addresses all the measures and procedures used by maintenance crews to remove or control vegetation on District right of way. These measures consist of the application of herbicides and other chemical control agents, mechanical removal, hand labor removal and agricultural methods. An EIR was prepared to assess the short-term, long-term, and cumulative impacts of this program on wildlife and fish habitat, surface water and groundwater quality, evaluate alternate means of accomplishing the goals of the program, and recommend mitigation to compensate for any adverse impacts.

- Erosion Control and Sediment Removal Routine Maintenance Program: This program addresses all measures and procedures used by the Maintenance Department to perform erosion control by various structural means and both instream and off-stream sediment removal. The purpose of this program is to develop a comprehensive approach to managing routine maintenance activities on District facilities. An EIR and related Corps permit applications were prepared to comply with Section 404 of the Clean Water Act. The EIR and permit applications assessed the short-term, long-term, and cumulative impacts of this program on wildlife and fish habitat; assessed surface water and groundwater quality impacts; evaluated a broad range of alternatives in an equal level of detail; reduced the level of maintenance on any stream, up to a point where the stream can be self-maintaining; and defined a compensatory mitigation for impacts.

- Summer Dams Fisheries Study: This project is to monitor and study the impacts of construction and operation of in-stream seasonal spreader dams on anadromous fish. Seasonal dams located in Coyote Creeks, Guadalupe River, Los Gatos Creek, Saratoga Creek, Upper Penitencia Creek, and Stevens Creek are included in the study.

Community Projects Review

The District has adopted policies and ordinances to regulate and protect the water resources in the County. Ordinance 83-2 applies to all water resources in the County and prohibits the pollution of the water supplies of the District, whether in surface streams, reservoirs, other conduits of any kind, or groundwater. This ordinance requires a District permit for the installation of any outfall discharging into a watercourse under District jurisdiction, revegetation, and any construction or soil disturbances within 50 feet of any creek bank. The objective of Ordinance 83-2 is to allow the District to properly manage streams to prevent or minimize erosion, preserve water quality, contribute to survival of the area's fish and wildlife, and help avoid flood losses. The life of a stream reflects conditions throughout its watershed, and not just those along its banks. The District's Design Coordination Division reviews plans and issues a permit only after investigating that the proposed work will not impact the District's facilities. Violation of the provisions of the ordinance and any failure to satisfy the terms of the permit are punishable as a misdemeanor.

Some of the programs with which the Community Projects Review Unit is actively involved in are:

- Ordinance 83-2 Violations Program: This program identifies violations of Ordinance 83-2 throughout the County and requires the violator to correct the condition within a specified time. Examples of common violations are: encroachment, illegal outfall, dumping, and erosion. In the Guadalupe River watershed, approximately 56 violations were reported and acted upon by the District in 1993.

- Stream Care Brochure: The brochure promotes public awareness and encourages active participation from every resident in the County. This brochure addresses the preservation and enhancement of the streamside environment, protection of streams, and effective watershed management. The brochure targets streamside property owners to help reduce the number of Ordinance 83-2 violations.

Water Supply Programs

As a water wholesaler and manager of water resources in the County, the District conserves, imports, treats, distributes, reclaims, and is responsible for the quality of water in the County. To meet its water supply objectives, the District constructed and now operates and maintains a countywide conservation and distribution system to convey raw water for recharge and treatment.

- Reservoir Storage: The District currently operates and maintains ten local reservoirs which conserve local runoff for either local recharge or treatment at one of the District's three water treatment plants. Five of the ten local water conservation reservoirs are within the Guadalupe River watershed. The Calero/Almaden reservoir systems have the ability to deliver raw water to one of three water treatment plants.
- Artificial Groundwater Recharge Program: The District owns and operates over 30 recharge facilities in 6 major recharge systems. These recharge facilities percolate local or imported water into the groundwater basin. The Los Gatos and Guadalupe recharge systems are within the Guadalupe River watershed). In-stream recharge is enhanced by the construction and operation of temporary dams. Off-stream recharge is performed at percolation ponds that are fed by water diverted from the creeks or by imported water pipelines.

City of San Jose

The San Jose City Council adopted the Horizon 2000 General Plan for the City in 1984 and Horizon 2020 in 1994. The Plan seeks to balance natural resource conservation and urban development so as to maximize the achievement of environmental, economic, and social needs.

The City plans to protect water resources wherever possible. Policy No. 3 of the City's Water Resources Goals and Policies states: "Protect water resources because they are vital to ecological and economic health of the region and its residents. The City, in cooperation with the District, should restrict, or carefully regulate, public and private development in watershed areas, especially those necessary for effective functioning of reservoirs, ponds, and streams, and for the prevention of excessive siltation. The City should protect important groundwater recharge areas, particularly creeks and creeksides, from urban development."

The City also plans to preserve riparian resources wherever feasible. Policy No. 5 of the City's Marinelife and Wildlife Resources goals and Policies states: "Preserve areas of special marine and wildlife habitation, particularly those containing endangered species, as living research and recreational resources, and as indispensable parts of the total environment. Any development involving unmitigated adverse impacts on the National Wildlife Refuge in South San Francisco Bay should not be permitted. Consideration should be given to setting aside nature preserves in the Bay, marshes, streamsides, streambeds, and hillside areas to protect habitats of essential, rare and endangered species of plants and animals, and to provide areas for educational and research purposes."

The City has adopted a riparian corridor policy which balances long-term environmental protection and continued land development. This policy provides the City information to identify and manage its riparian resources in an environmentally sensitive manner to protect them for environmental as well as recreational purposes. The policy identifies specific development guidelines:

> "Incompatible operations and activities are discouraged within and adjacent to riparian setback areas to protect the health of existing vegetation and wildlife, reduce adverse cumulative impacts to water quality, and protect the quality of recreation uses in the corridor."
>
> "Development adjacent to riparian habitats generally should be set back 100 feet from the outside edge of the riparian habitat (or top of bank, whichever is greater) to reduce anticipated impacts to riparian biotic communities and hydrologic regimes."
>
> In areas where the creek channel is deeply incised and banks are unstable, actively eroding, or identified as hazardous to public safety, banks should be stabilized/protected using biotechnical bank protection measures (e.g., riparian plantings, vegetated gabions or vegetated cribwalls), consistent with flood control concerns. The use of hard bank structures, such as concrete walls/slopes, sackcrete and unvegetated rip-rap or gabions is discourged."
>
> "Where gabions, flood walls, and other armoring materials and techniques are necessary for flood protection and slope protection, planting pockets and terraces should be created as an integral part of the structures. Native vegetation should be established and maintained in the plantikng pockets and terraces."
>
> "Bypass channels or culverts are the preferred methods to improve flood flows and channel capacity. Bypass channels/culverts should be designed to preserve riparian areas on both sides of the existing channel."

The City zoning regulations are intended to implement the General Plan by providing standards for development. The various ranges of allowed land uses and development intensity within a given zoning district correspond generally to the respective General Plan land use designations. Although the conventional zoning districts contained in the City's Zoning Ordinance include an Open Space zoning district that may be applied to properties designated for open space use by the General Plan, this Open Space zoning district permits certain uses which are compatible with open space generally but may be inappropriate in riparian corridors. Planned Development zoning is also available to tailor allowed uses, site intensities, and development standards to a particular site. Planned Development zoning could be used to implement standards appropriate for the protection of riparian areas on a site-specific basis. The Zoning Ordinance also regulates development through a number of development permits intended to implement the General Plan. Appropriate treatment of riparian corridors also can be achieved through these permit processes.

The City's Subdivision ordinance implements the State Subdivision Map Act by requiring all subdivisions in the City to be consistent with the City's General Plan and state law. The State Subdivision Map Act requires that "reasonable public access" to be provided and requires the dedication of a public easement along the banks of rivers or streams bordering or within a proposed subdivision, unless the City makes a finding that alternative public access is otherwise available within a reasonable distance. This public access easement could be implemented to include riparian areas on a site-specific basis.

The City is a participant in the Santa Clara Valley Nonpoint Source Pollution Control Program and a co-permittee with 14 other agencies in the County for the NPDES permit issued by EPA to the program in June of 1990. As a participant in the Nonpoint Source Program, the City is responsible for implementing program elements in its area of jurisdiction and must submit an annual report to be included in the Santa Clara Valley NPDES annual report.

Santa Clara County

The County General Plan (1990) has a few protective measures in reference to creeks and streamside areas in its Natural Environment, Lands Use and Public Safety Elements. Some of them are:

- Public projects shall be designed to avoid damage to the stream environments.
- Where possible, riparian woodlands, marshes and floodplains which have been altered should be allowed to return to a natural state.
- In floodplains which are not already developed, land uses shall be restricted to avoid need for major flood control alterations to the streams.
- Public projects should preserve the stream environment and should provide multiple use for such purposes as parks, open space preserves, trails and flood control.

State Regulations

California Environmental Quality Act

The California Environmental Quality Act (CEQA) is California's broadest environmental law.

There are six objectives of CEQA:

- To disclose to decision makers and the public the significant environmental effects of proposed activities.
- To identify ways to avoid or reduce environmental damage.
- To prevent environmental damage by requiring implementation of feasible alternatives or mitigation measures.
- To disclose to the public reasons for agency approvals of projects with significant environmental effects.
- To foster interagency coordination.
- To enhance public participation.

The fundamental premise on which CEQA is based is that environmental protection can be achieved through compliance with rigorous, action-forcing procedures. CEQA also contains substantive provisions requiring agencies to avoid or mitigate impacts disclosed in an environmental impact report (EIR) when feasible. CEQA grants public agencies five basic types of authority to enable them to carry out the objectives of the law:

- To require changes in a project to lessen or avoid significant effects, when feasible.
- To disapprove a project to avoid significant effects.
- To approve a project with unavoidable significant effects if the project's benefits outweigh these effects.
- To comment on other agencies' environmental documents.
- To impose fees on project applicants for CEQA implementation.

Streambed Alteration Agreements: California Fish and Game Code Section 1601/1603

Any person, governmental agency, state or local, and public utility proposing a project which will diverts, obstructs or change the natural flow or bed, channel, or bank of any river, stream or lake must give notice to the CDFG under Sections 1601 (public) and 1603 (private) of the California Fish and Game Code.

California State Water Resources Control Board

The California State Water Resources Control Board (SWRCB) is responsible for the quality of the water in the state waterways. The nine RWQCBs that implement the policies of the SWRCB, make policy recommendations and issue permits to improve water quality. The San Francisco Regional Board has issued three general NPDES permits which are applicable in the Guadalupe River watershed.

NPDES General Industrial Activity Stormwater Permit

This permit applies to storm water discharges by certain regulated industries. Federal law requires that industrial storm water discharges meet all provisions of Section 301 and 402 of the Federal Clean Water Act in order to control pollutant discharges. These provisions require the use of best available technology economically available and best conventional pollution control technology to reduce pollutants and any more stringent controls necessary to meet water quality standards.

NPDES General Construction Activity Stormwater Permit

This permit is required of all construction projects disturbing five or more acres of land area. BMPs to control storm water runoff both during and after construction are required.

NPDES Municipal Stormwater Permit

Municipalities with a population over 100,000, or those that have been determined to be significant contributors of pollutants, are required to obtain NPDES municipal storm water permits. The applicants are required to collect existing information regarding storm water discharges and receiving water quality, provide adequate fiscal resources, and prepare a storm water management program. The Guadalupe River watershed falls within the jurisdiction of the NPDES permit issued to the Santa Clara Valley Nonpoint Source Pollution Control Program.

Federal Regulations

The Rivers and Harbors Act

The Rivers and Harbors Act was enacted by Congress in 1899 to protect interstate commerce in navigable waters through the regulation of streams and rivers. Sections 9 and 10 of the Act relate to protecting navigable waters. Section 9 of the Rivers and Harbors Act requires an applicant to obtain a permit to construct a dike or dam in navigable waters of the United States. Under Section 10, the Corps regulates projects or construction of structures in or over any navigable water of the United States, including the excavating from or depositing of material in any such waters. Limits of navigable water jurisdiction in Guadalupe River extend up through the upstream end of this project.

Fish and Wildlife Coordination Act

The Fish and Wildlife Coordination Act was established to protect the quality of the aquatic environment as it affects the conservation, improvement and enjoyment of fish and wildlife resources. The Fish and Wildlife Coordination Act requires that any federal agency that proposes to control or modify any body of water must first consult with the U.S. Fish and Wildlife Service (USFWS) and the National Marine Fisheries Service (NMFS).

Endangered Species Act (Section 7)

The federal Endangered Species Act requires federal agencies, in consultation with the NMFS and the USFWS, to ensure that their actions do not jeopardize the continued existence of endangered or threatened species or result in the destruction or adverse modification of the critical habitat of these species.

The State Endangered Species Act requires agencies to consult formally with DFG when the proposed action may affect state-listed endangered or threatened species.

Clean Water Act

The Clean Water Act was enacted by Congress in 1972 and its goal was to "restore and maintain the chemical, physical, and biological integrity of the Nation's waters." To meet this objective, Section 301 of the Clean Water Act prohibits the discharge of any pollutants into navigable waters, except what is allowed by permits issued under Sections 402 and 404.

Sections 401, 402, and 404 of the Clean Water Act

Section 401 addresses effluent standards and water quality standards by requiring, in addition to a dredge and fill discharge permit, water quality certification from the SWRCB.

Section 402 authorizes the U.S. Environmental Protection Agency (USEPA) to issue permits under procedures established to implement the NPDES program.

Section 404 authorizes the Corps to issue permits for and regulate the discharge of "dredge or fill materials into waters of the United States." Waters of the United States include:

- all waters which are currently used, or which were used in the past for commerce, including waters which are subject to the tide;
- all interstate waters, including lakes, rivers, streams, intermittent streams, sloughs, and natural ponds;
- all impoundments of water;
- tributaries of waters; and
- wetlands adjacent to waters.

Once an area has been identified as a federal water and a permit is applied for, an environmental assessment is required under the National Environmental Policy Act. The NMFS and USFWS review and comment on all Section 404 permit applications. In addition, the RWQCB is responsible for certifying Section 404 permits as being in compliance with state water quality standards.

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APPENDICES

A. Upper Guadalupe River Flood Flow Reduction Study Urban Detention Alternative Design Cost Estimate Summary.

B. Excerpts from Guadalupe River Watershed Planning Study Engineers Report.

C. Excerpts from Summary of Dam and Reservoir Investigations in Santa Clara County.

D. Excerpts from Lexington Reservoir Enlargement Feasibility Study.

E. Excerpts from Guadalupe River Evaluation of Offstream Storage Alternative.

F. Excerpts from Study of Factors Influencing Erodability of Soils in Unlined Earth Channels in the Santa Clara Valley.

G. Sedimentation on South San Francisco Bay Below Dumbarton Bridge.

H. Santa Clara Valley Water District Urban Hydrology Use of the Corps of Engineers Flood Hydrograph Package as a Predictive Model.

UPPER GUADALUPE RIVER FLOOD FLOW REDUCTION STUDY

In-Line Urban Storage Basins Alternative Design
This is per square mile, 57,000 lf of enlarged storm drain to create 310 AF of on-line storage
Double 15x8 RCB Holding Basin 57000 feet long
Cost Estimate Summary

Item	Unit Cost	Unit	Quantity	Amount
Site Preparation				
Clearing and Grubbing (57,000x50)/43560	$5,500	AC.	195	$1,072,500
Remove Residential Sites	$7,500	EA.	0	$0
Remove Commercial Sites	$3	S.F.	0	$0
Coffer dams installation (fill for 4)	$7	C.Y.	0	$0
Coffer dams removal (excavation for 4)	$6	C.Y.	0	$0
Earthwork				
Excavation (40x10/27=14.8 cy per lf x 57000lf)	$6	C.Y.	845,000	$5,070,000
Hauling	$6	C.Y.	845,000	$5,070,000
Structural backfill (100/27=3.7 cy per lf x 57000)	$7	C.Y.	210,000	$1,470,000
Fill	$7	C.Y.	0	$0
Concrete Structures				
Structure Removal	$50,000	L.S.	0	$0
RCB Culvert. (84x1.25/27=3.9 cy per lf x57000)	$450	C.Y.	222,000	$99,900,000
Traffic Control	$300,000	mi	11	$3,240,000
Utility Relocation/Modification				
Water Lines Relocation	$200	L.F.	57,000	$11,400,000
Sanitary sewer relocation	$200	L.F.	57,000	$11,400,000
Outfalls	$5,000	EA.	0	$0
Roadway Replacement				
Remove A/C Pavement (50sf/lf x57000)	$2	S.F.	2,850,000	$5,700,000
Remove Curb & Gutter	$2	L.F.	0	$0
Remove Sidewalk	$2	S.F.	0	$0
Construct A/C Pavement(50sf/lf x57000)	$5	S.F.	2,850,000	$14,250,000
Construct Curb & Gutter	$16	L.F.	0	$0
Construct Sidewalk	$6	S.F.	0	$0
Miscellaneous				
Armoring Aggregate. (6in. on Maint. Road)	0.55	S.F.	0	$0
Gates	$800	EA.	0	$0
Fencing	$10	L.F.	0	$0

Item	Unit Cost	Unit	Quantity	Amount
SUBTOTAL				$158,572,500
Mobilization (10%)				$15,857,250
Construction Contingencies (10%)				$15,857,250
TOTAL CONSTRUCTION COSTS				**$190,287,000**
Design (10%)				**$19,028,700**
Geotechnical (5%)				**$9,514,350**
Inspection (10%)				**$19,028,700**
Property Acquisition (per Real Estate cost estimate)				
Residential				$0
RIGHT-OF-WAY COSTS				**$0**
Mitigation				
Landscaping (on-site 10 ft strip)	$27	L.F.	57,000	$1,539,000
Revegetation (off-site)	$200,000	AC.	0	$0
Revegetation right of way (off-site)	$500,000	AC.	0	$0
TOTAL COSTS				$239,397,750

Cost is per square mile of urban developed area

UPPER GUADALUPE RIVER FLOOD FLOW REDUCTION STUDY

Retrofit with On-Site Storage Basin Alternative Design
This is per parcel, 5300 cubic feet of enlarged subterranean storage
Subterrainean storage 33X33x5 feet deep
Cost Estimate Summary

Item	Unit Cost	Unit	Quantity	Amount
Site Preparation				
Clearing and Grubbing	$1,000	EA.	1	$1,000
Raise structure during construction	$15,000	EA.	1	$15,000
Remove Residential Sites	$7,500	EA.	0	$0
Remove Commercial Sites	$3	S.F.	0	$0
Coffer dams installation (fill for 4)	$7	C.Y.	0	$0
Coffer dams removal (excavation for 4)	$6	C.Y.	0	$0
Earthwork				
Excavation (33x33x5/27=202 cy)	$12	C.Y.	202	$2,424
Hauling	$6	C.Y.	202	$1,212
Structural backfill	$7	C.Y.	0	$0
Fill	$7	C.Y.	0	$0
Concrete Structures				
Structure Removal	$50,000	L.S.	0	$0
Concrete basin (2838/27=105 cy)	$450	C.Y.	105	$47,250
Traffic Control	$300,000	mi	0	$0
Utility Relocation/Modification				
Water Lines Relocation	$1,000	EA.	1	$1,000
Sanitary sewer relocation	$10,000	EA.	1	$10,000
Outfall pipe to stormdrain	$15,000	EA.	1	$15,000
Roadway Replacement				
Remove A/C Pavement	$2	S.F.	0	$0
Remove Curb & Gutter	$2	L.F.	10	$20
Remove Sidewalk	$2	S.F.	40	$80
Construct A/C Pavement	$5	S.F.	0	$0
Construct Curb & Gutter	$16	L.F.	10	$160
Construct Sidewalk	$6	S.F.	40	$240
Miscellaneous				
Armoring Aggregate. (6in. on Maint. Road)	0.55	S.F.	0	$0
Gates	$800	EA.	0	$0

Item	Unit Cost	Unit	Quantity	Amount
Fencing	$10	L.F.	0	$0
SUBTOTAL				$93,386
Mobilization (10%)				$9,339
Construction Contingencies (10%)				$9,339
TOTAL CONSTRUCTION COSTS				**$112,063**
Design (10%)				**$11,206**
Geotechnical (5%)				**$5,603**
Inspection (10%)				**$11,206**
Property Acquisition (per Real Estate cost estimate)				
Residential				$0
RIGHT-OF-WAY COSTS				**$0**
Mitigation				
Replace Landscaping	$10,000	EA	1	$10,000
Revegetation (off-site)	$200,000	AC.	0	$0
Revegetation right of way (off-site)	$500,000	AC.	0	$0
TOTAL COSTS				**$150,079**

Cost is per parcel

UPPER GUADALUPE RIVER FLOOD FLOW REDUCTION STUDY

Retrofit with On-Site Storage Basin Alternative Design
New Storm drain collector for individual parcel basins
There are 31.7 miles(167,000 ft) of roads (30% of area with avg 50 wide) per square mile of urban area
There are 167,000 - 57,000 LF which need new stormdrain collector in roadways
Cost Estimate Summary

Item	Unit Cost	Unit	Quantity	Amount
Site Preparation				
Clearing and Grubbing	$1,000	AC.	0	$0
Remove Residential Sites	$7,500	EA.	0	$0
Remove Commercial Sites	$3	S.F.	0	$0
Coffer dams installation (fill for 4)	$7	C.Y.	0	$0
Coffer dams removal (excavation for 4)	$6	C.Y.	0	$0
Earthwork				
Excavation (3x8x110000/27=97000 cy)	$6	C.Y.	97,000	$582,000
Hauling	$6	C.Y.	97,000	$582,000
Structural backfill	$7	C.Y.	30,000	$210,000
Fill	$7	C.Y.	0	$0
Concrete Structures				
Structure Removal	$50,000	L.S.	0	$0
Concrete	$450	C.Y.	0	$0
Traffic Control	$300,000	mi	0	$0
Utility Relocation/Modification				
Water Lines Relocation	$200	LF	0	$0
Sanitary sewer relocation	$200	LF	0	$0
Install new stormdrain 110,000 LF	$200	LF	110,000	$22,000,000
Traffic Control	$300,000	mi	20	$6,000,000
Outfalls	$5,000	EA.	10	$51,000
Roadway Replacement				
Remove A/C Pavement (3sf/lf x110000)	$2	S.F.	330,000	$660,000
Remove Curb & Gutter	$2	L.F.	10	$20
Remove Sidewalk	$2	S.F.	40	$80
Construct A/C Pavement(3sf/lf x110000)	$5	S.F.	330,000	$1,650,000
Construct Curb & Gutter	$16	L.F.	10	$160
Construct Sidewalk	$6	S.F.	40	$240
Miscellaneous				

Item	**Unit Cost**	**Unit**	**Quantity**	**Amount**
Armoring Aggregate. (6in. on Maint. Road)	0.55	S.F.	0	$0
Gates	$800	EA.	0	$0
Fencing	$10	L.F.	0	$0
SUBTOTAL				$31,735,500
Mobilization (10%)				$3,173,550
Construction Contingencies (10%)				$3,173,550
TOTAL CONSTRUCTION COSTS				**$38,082,600**
Design (10%)				**$3,808,260**
Geotechnical (5%)				**$1,904,130**
Inspection (10%)				**$3,808,260**
Property Acquisition (per Real Estate cost estimate)				
Residential				$0
RIGHT-OF-WAY COSTS				**$0**
Mitigation				
Landscaping	$120,000	AC.	0	$0
Revegetation (off-site)	$200,000	AC.	0	$0
Revegetation right of way (off-site)	$500,000	AC.	0	$0
TOTAL COSTS				$47,603,250

Cost is per square mile of urban developed area

SANTA CLARA VALLEY WATER DISTRICT

GUADALUPE RIVER WATERSHED PLANNING STUDY

ENGINEER'S REPORT

PROJECT NO. 3015

Engineer's Report Prepared by Flood Management and Policy Planning Unit

Dennis Cheong, P.E.	Associate Civil Engineer
Afshin Rouhani	Assistant Civil Engineer
Randall R. Talley, P.E.	Supervising Engineer

(Environmental Impact Report—Bound Separately)
Prepared Under the Direction of

Terrence A. Neudorf
Environmental Planner
Environmental Resources Management Unit

by
Parsons Engineering-Science, Inc.

Stanley M. Williams	James W. Ferguson
General Manager	Executive Project Manager

NOVEMBER 1999

DISTRICT BOARD OF DIRECTORS

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Adverse impacts of the proposed design on either creek will be mitigated. The impacts will be determined by comparing the existing conditions to future design conditions on Guadalupe River. The design 1 percent flow rates of Ross and Canoas Creeks are shown in Table 3.5.

TABLE 3.5
Ross Creek and Canoas Creek Design Flow Rates (cfs)

Location	1%	10%
Ross Creek at Meridian Avenue	1,800	1,200
Ross Creek Upstream Guadalupe River	2,200	1,300
Canoas Creek at Capitol Expressway	2,000	1,900
Canoas Creek Upstream Guadalupe River	2,400	2,000

ALTERNATIVES FEASIBILITY AND THEIR SIGNIFICANT IMPACTS

The alternatives considered can be divided into six general categories and are summarized in Table 3.6.

"No Project" Alternative

Under the "no project" alternative, the District would not take any action on the Guadalupe River except routine maintenance work, which is done on a priority basis but without an overall plan for the creek. Residents and businesses within the floodplain would continue to face potential hardships due to flooding. Private properties immediately adjacent to the creek would continue to be at risk from stream bank failure and debris blockage. Flooding could still be expected to occur on the average of about once every 4 years. Ongoing active erosion could be expected to continue and possibly damage adjacent properties and trees within the riparian corridor. Water quality and creek environment will continue to degrade. Implementing future park and trail master plans will be difficult. Fish passage for upstream migration will not be restored. Annual maintenance and erosion repair costs could be expected to continue at about $200,000 each year as they have been the last several years.

In the event of the 1 percent flood, approximately 2,200 acres would be inundated. Over 7,200 homes, 230 businesses, 11 public buildings, and 1,390 automobiles would be inundated by floodwaters from Guadalupe River. Sediment deposition on roadways would result in additional cleanup costs. One percent flood damages of over $280 million (1995 present worth) could be expected. Average annual damages would be an estimated $20.6 million per year.

Homeowners and businesses residing in the 1 percent floodplain, where flood depths are 1-foot or greater, would still be required to purchase flood insurance through the NFIP[1]. Insurance premium costs to the owners of the 3,850 structures covered by the program would be over $1 million annually.

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[1] Mandatory participation in the program is required if the owner has a federally insured loan, Public Law 93-234, the "Flood Protection Act of 1973."

New development within the floodplain would require flood-proofing of the structures. There are more than 160 acres in the floodplain which are expected to be developed by the year 2000[2]. The estimated incremental new development cost for flood-proofing is nearly $3 million

The "no project" alternative is considered infeasible and impracticable because of its high cost in flood damages and it does not solve the significant flooding, erosion, water quality, and creek environment degradation problems.

Nonstructural Plan

A nonstructural alternative would solve the flooding of structures in the floodplain without channel modifications. The structures would either be flood proofed or purchased and removed. Regulating floodplain development does not apply because the floodplain is already well developed. Flooding would still occur on the average of every 4 years causing disruption in services, street flooding, and resulting in clean up costs.

Erosion would be expected to continue, requiring repair when structures are threatened. Water quality and creek environment will continue to degrade. Implementing a future park and trail master plan will be difficult. Fish passage for upstream migration will not be restored.

This alternative is considered infeasible and impracticable due to the high cost of flood proofing over 7,200 structures in the floodplain (over $600 million), the social impact of relocating families and removing homes, the continuing erosion and the residual damages from flooding, and the continuing water quality and creek environment degradation.

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[2] Based on the "Horizon 2000 General Plan" adopted by the City of San Jose, November 7, 1984.

TABLE 3.6
Guadalupe River—Summary of Conceptual Alternatives

Problems, Concerns, and Impacts	Conceptual Alternatives						
	Existing Conditions	No Project	Nonstructural	Upstream Reservoirs Construction	Change Upstream Reservoirs Operation	Offstream Storage	Channel Modification Only
Description Conceptual Alternatives	Major problems are water quality degradation and lack of adequate capacity to carry 1 percent design flow. Other significant problems are erosion, sedimentation, local drainage, degraded fish habitat, fish passage, and maintenance problems.	No measures taken to reduce current risk of flooding. Mandatory flood insurance required in floodprone areas. Ongoing erosion would be repaired on an emergency basis. Existing condition would remain if not worsen.	Flood damage to structures would be solved without channel modifications. 7,200 structures would be floodproofed. Regulate floodplain development does not apply because the floodplain is already well developed. Ongoing erosion would be repaired on an emergency basis. Existing condition would remain.	Construct new Guadalupe Dam and enlarge Almaden Dam to reduce peak flow rate downstream. Significant channel modifications would also be required in Reaches 6 to 10A. Mitigate the impacts on site and on adjacent lands.	Alter operation of three water supply reservoirs to flood control. Significant channel modifications would also be required in Reaches 6 to 10A. Mitigate the impacts on site and on adjacent lands.	Build diversion structure and offstream detention ponds to reduce peak flow rate in channel. Significant channel modifications would also be required in Reaches 6 to 10A. Mitigate the impacts on site and on adjacent lands.	Modify existing stream channel to contain the 1 percent flood and resist scour from high velocities. Mitigate the impacts on site and on adjacent lands.
Flooding Problems	High risk of flooding, continue mandatory flood insurance.	Existing condition would remain. Expected annual flood damages of $20.6 million.	Flooding would still occur causing some damages, disruption of services and resulting in cleanup costs.	Reservoirs alone would not eliminate the threat of 1 percent flood and the mandatory flood insurance. Additional channel modifications would provide 1 percent protection.	Change of reservoirs operation alone would not eliminate the threat of 1 percent flood and the mandatory flood insurance. Additional channel modifications would provide 1 percent protection.	Offstream ponds alone would not eliminate the threat of 1 percent flood and mandatory flood insurance. Additional channel modifications would provide 1 percent protection.	Eliminate the threat of 1 percent flood and mandatory flood insurance.
Maintenance Concerns	Poor maintenance access, requires intense maintenance. Discontinuous right of way and easements	Existing condition would remain.	Existing condition would remain.	Improve maintenance access, requires less intense maintenance in Reaches 6 to 10A.	Improve maintenance access, requires less intense maintenance in Reaches 6 to 10A, but more at reservoirs.	Improve maintenance access, requires less intense maintenance in Reaches 6 to 10A.	Improve maintenance access, requires less intense maintenance in Reaches A to 12.
Erosion Problems	Active erosion occurs due to steep and unstable banks.	Existing condition would remain if not worsen. Erosion repair on priority basis.	Existing condition would remain. Erosion repair on priority basis.	Repair areas of active erosion, decreases erosion potential in Reaches 6 to 10A.	Repair areas of active erosion, decreases erosion potential in Reaches 6 to 10A.	Repair areas of active erosion, decreases erosion potential in Reaches 6 to 10A.	Repair areas of active erosion, decreases erosion potential in Reaches A to 12.

TABLE 3.6
Guadalupe River—Summary of Conceptual Alternatives
(continued)

Problems, Concerns, and Impacts	Conceptual Alternatives						
	Existing Conditions	**No Project**	**Nonstructural**	**Upstream Reservoirs Construction**	**Change Upstream Reservoirs Operation**	**Offstream Storage**	**Channel Modification Only**
Sedimentation Problems	Periodic removal of sediment in the channel bottom.	Existing condition would remain.	Existing condition would remain.	Requires less frequent sediment removal in Reaches 6 to 10A.	Requires less frequent sediment removal in Reaches 6 to 10A.	Requires less frequent sediment removal in Reaches 6 to 10A.	Requires less frequent sediment removal in Reaches A to 12.
Local Drainage Problems	Long duration and high frequency of flooding in the streets and local ponding.	Existing condition would remain.	Existing condition would remain.	The duration and frequency of flooding in the streets and local ponding would decrease slightly.	The duration and frequency of flooding in the streets and local ponding would decrease slightly.	The duration and frequency of flooding in the streets and local ponding would decrease slightly.	The duration and frequency of flooding in the streets and local ponding would decrease slightly.
Water Quality Problems	Degraded water quality.	Existing condition would remain if not worsen.	Existing condition would remain if not worsen.	Some improvement in water quality due to improved creek environment in Reaches 6 to 10A.	Some improvement in water quality due to improved creek environment in Reaches 6 to 10A.	Some improvement in water quality due to improved creek environment in Reaches 6 to 10A.	Improvement in water quality due to improved creek environment in Reaches 6 to 12.
Water Supply Impacts	Limited water supply potential.	Existing condition would remain.	Existing condition would remain.	Increase water storage capacity in the summer months at reservoirs.	Loss of 13,900 acre-foot/year because reservoirs would be kept empty during the winter months at the cost of $5 million/year. Supplemental water may not be readily available for purchase. There may be regulatory constraints in the federal and state systems that impact the ability to import water to the county.	Increase groundwater recharge potential in the summer months at the off-stream ponds.	Small increase groundwater recharge potential at Reach 12.

TABLE 3.6
Guadalupe River—Summary of Conceptual Alternatives
(continued)

Problems, Concerns, and Impacts	Conceptual Alternatives						
	Existing Conditions	No Project	Nonstructural	Upstream Reservoirs Construction	Change Upstream Reservoirs Operation	Offstream Storage	Channel Modification Only
Riparian Vegetation Impacts	Degraded and discontinuous riparian corridor.	Existing condition would remain if not worsen.	Existing condition would remain if not worsen.	Major impacts to 1.6 creek miles and 167 acres of riparian vegetation at reservoir sites. Additional impacts to 2.7 creek miles from channel modifications. Impacts mitigated partially on site and on adjacent lands would reestablish a continuous riparian corridor along the stream in Reaches 6 to 10A.	Impacts mitigated on site and on adjacent lands would reestablish a continuous riparian corridor along the stream in Reaches 6 to 10A.	Impacts mitigated on site and on adjacent lands would reestablish a continuous riparian corridor along the stream in Reaches 6 to 10A.	Impacts mitigated on site and on adjacent lands would reestablish a continuous riparian corridor along the stream in Reaches A to 12.
Wetland Impacts	Moderate wetland habitat.	Existing condition would remain if not worsen.	Existing condition would remain if not worsen.	Major impacts to wetlands at reservoir sites and from channel modifications. Impacts can only be partially mitigated in Reaches 10 and 12.	Major impacts to wetlands at reservoir sites and from channel modifications. Impacts can only be partially mitigated in Reaches 10 and 12.	Mitigate by creating wetlands in Reaches 10 and 12.	Mitigate by creating wetlands in Reaches 10 and 12.
Wildlife Habitat Impacts	Moderate wildlife habitat.	Existing condition would remain if not worsen.	Existing condition would remain if not worsen.	Major impacts to wildlife habitat at reservoir sites and from channel modifications. Impacts mitigated partially on site and on adjacent lands would reestablish a continuous wildlife corridor along the stream in Reaches 6 to 10A.	Major impacts to wildlife habitat at reservoir sites and from channel modifications. Impacts mitigated partially on site and on adjacent lands would reestablish a continuous wildlife corridor along the stream in Reaches 6 to 10A.	Impacts mitigated on site and on adjacent lands would reestablish a continuous wildlife corridor along the stream in Reaches 6 to 10A.	Impacts mitigated on site and on adjacent lands would reestablish a continuous wildlife corridor along the stream in Reaches A to 12.

TABLE 3.6
Guadalupe River—Summary of Conceptual Alternatives
(continued)

Problems, Concerns, and Impacts	Conceptual Alternatives						
	Existing Conditions	**No Project**	**Nonstructural**	**Upstream Reservoirs Construction**	**Change Upstream Reservoirs Operation**	**Offstream Storage**	**Channel Modification Only**
Fishery Impacts	Degraded fish habitat, existing fish barriers prevent fish from migrating upstream, limited low flow channel.	Existing condition would remain if not worsen.	Existing condition would remain if not worsen.	Major impacts to fish habitat at reservoir sites and from channel modifications. Impacts mitigated partially on site and on adjacent lands would reestablish a continuous low flow channel in Reaches 6 to 10A.	Major impacts to fish habitat at reservoir sites and from channel modifications. Loss of water releases from reservoirs in summer months. Impacts mitigated partially on site and on adjacent lands would reestablish a continuous low flow channel in Reaches 6 to 10A.	Impacts mitigated on site and on adjacent lands would reestablish a continuous low flow channel in Reaches 6 to 10A.	Impacts mitigated on site and on adjacent lands would reestablish a continuous low flow channel in Reaches A to 12, and remove all fish barriers and provide fish passage to upstream habitat.
Aesthetic Impacts	Degraded aesthetic qualities due to concrete rubble and sacked concrete used for erosion repair, illegal dumping, nonpoint source pollution.	Existing condition would remain if not worsen.	Existing condition would remain if not worsen.	Implement better environmental approach to erosion repair work, provide better maintenance access for cleanup and improve creek environment in Reaches 6 to 10A.	Implement better environmental approach to erosion repair work, provide better maintenance access for cleanup and improve creek environment in Reaches 6 to 10A.	Implement better environmental approach to erosion repair work, provide better maintenance access for cleanup and improve creek environment in Reaches 6 to 10A.	Implement better environmental approach to erosion repair work, provide better maintenance access for cleanup and improve creek environment in Reaches A to 12.
Recreation Impacts	No access to the river corridor, no recreation potential.	Existing condition would remain.	Existing condition would remain.	Increase recreation area at reservoirs during the summer months in Reaches 6 to 10A.	Increase recreation area at reservoirs during the summer months in Reaches 6 to 10A.	Increase recreation area around the ponds in Reaches 6 to 10A.	Maintenance access and roads are designed to allow for future trail development connecting Los Alamitos to downtown San Jose.
Cost as 1995 Present Worth		$280 million in flood damages	$600 million	$173 million	$120 million	$600 million	$114.0 million
Time to Complete Project				50 years	40 years		25 years

TABLE 3.6
Guadalupe River—Summary of Conceptual Alternatives
(continued)

Problems, Concerns, and Impacts	Conceptual Alternatives						
	Existing Conditions	No Project	Nonstructural	Upstream Reservoirs Construction	Change Upstream Reservoirs Operation	Offstream Storage	Channel Modification Only
Summary		Not selected because it does not solve the significant flooding, erosion, water quality, and creek environmental degradation problems.	Not selected due to high cost. Residual damages from flooding continued. It does not solve the significant water quality and creek environmental degradation problems.	Not selected because of the substantial impact to the riparian ecosystem and the high cost of construction.	Not selected because of substantial adverse impact on availability of water and the uncertainty of relying on additional imported water due to increasingly stringent protection measures for the environment in the Delta.	Not selected because of high cost.	Selected because it achieves most of the District's goals and objectives.

Upstream Storage (Reservoir Construction)

Three upstream storage alternatives were investigated by the Corps in the 1975 Phase I Study Report (20). Two of the alternatives were on the Los Gatos Creek and would have only benefitted the Guadalupe River downstream of the confluence with Los Gatos Creek.

The third alternative proposed construction of a new 50-foot high dam on Guadalupe Creek approximately 2 miles downstream of the existing Guadalupe Reservoir with a capacity over 2,000 acre-feet (see Figure 3.2). The "New Guadalupe Dam" alternative was investigated and the Corps determined that such a measure would only reduce the 1 percent flow rate by 1,700 cfs, which is not sufficient to avoid extensive channel modifications downstream on the Guadalupe River. The alternative was eliminated from further consideration by the Corps because of the low benefit-to-cost ratio (.33).

In 1978, the District investigated the feasibility of modifying the existing Almaden Reservoir. Of the three reservoirs on upstream tributaries, Almaden has the largest drainage area (12 square miles) but the smallest storage capacity (1,780 acre-feet).

By raising the existing Almaden Dam by 50 feet, a reservoir with a total storage capacity of 4,400 acre-feet would be created. The additional 2,620 acre-feet would reduce the 1 percent flow by about 1,800 cfs on the Guadalupe River at Blossom Hill Road. Therefore, the combination of raising the Almaden Dam and constructing the "New Guadalupe Dam" would reduce the Guadalupe River 1 percent flow rate by about 3,500 cfs at Blossom Hill Road. This would still require extensive channel modifications in some of the downstream reaches on Guadalupe River and the acreage of impacts on the existing riparian vegetation would be significant.

This alternative would inundate the stream corridor environment for 1.6 miles and the adjacent riparian vegetation totaling 76 acres at the "New Guadalupe Dam" site. Raising Almaden Dam would inundate an additional 91 acres of riparian vegetation.

The cost estimates for construction of the new Guadalupe and raising Almaden Dams are $25 million and $33 million respectively. The additional cost of channel modifications in Reaches 6–10A would be about $50 million making the total construction cost of this alternative an estimated $108 million. The additional mitigation cost is estimated at $65 million. This alternative assumes that mitigation lands are available.

This alternative is considered infeasible and impracticable because (1) the reduction in flow rate is not sufficient to eliminate the need for channel modifications and associated impacts in the study area, (2) the substantial inundation to the riparian ecosystem at the dam sites, and (3) the cost of construction and mitigation is too high. Creek environment from Reaches 10 to 12 will continue to degrade. Implementing a future parks and trail master plan will be difficult. Fish passage for upstream migration will not be restored.

Upstream Storage (Reservoir Operation)

Another upstream storage alternative investigated was to modify the existing reservoir operations for temporary attenuation of flood peak flow through flood-routing.

This alternative proposed to modify the outlets at Almaden, Calero, and Guadalupe Reservoirs and keep them empty in the winter months to maximize flood capacity. The existing reservoirs do not have sufficient capacity and are not strategically located to have a significant impact on the downstream flood hydrograph. The watershed area in the study reach ranges from 53.2 square miles in Reach 12 to 95.3 square miles in Reach 6 and the drainage area for all three reservoirs totals less than 25.0 square miles.

This alternative would only reduce the Guadalupe River 1 percent flow by 2,200 cfs which would still require extensive channel modifications in the downstream reaches on the Guadalupe River. The acreage of impacts on existing riparian vegetation would be significant. (The additional cost of channel modifications in Reaches 6-10A is estimated for $60 million.)

This alternative would cause a loss of 13,900 acre-feet/year in local water supply yield which would need to be replaced at approximately $350/acre-feet (AF), an annual cost of $4,865,000/year. The water might not be readily available for purchase and there may be hydraulic and/or regulatory constraints in the federal and state system that impacts the ability to import the water to the county. Furthermore, the purchase of significant imported water may have significant impacts on the area of export, the bay and delta.

This alternative is considered infeasible and impracticable (1) the reduction in flow is not sufficient to eliminate required channel modifications and associated impacts in the study area; (2) the three existing reservoirs currently supplying 13,900 AF/year of water to Santa Clara County would have to be kept empty in the winter months and this water would be considered lost; (3) affects District's ability to store water that will be used during drought period; (4) may forego water rights, which contradicts the District's goal of making full use of economically feasible local water supplies; (5) increases the county's projected water needs at the year 2020; and (6) the uncertainty of relying on additional imported water due to increasingly stringent protection measures for the environment in the Delta. This would also have a substantial adverse effect on the availability of water and, therefore, fishery between May and September. Creek environment from Reaches 10 to 12 will continue to degrade. Implementing a future park and trail master plan will be difficult. Fish passage for upstream migration will not be restored.

A hydrologic analysis was performed to compare the three existing reservoirs operation with the natural runoff characteristics of the Guadalupe River watershed and how each case met predetermined flow requirements for the fishery resource. The results showed that the reservoir operation has very little or no adverse effect as it only altered slightly the natural runoff characteristics of the watershed. If greater flows were released in the wetter months it would not significantly change or enhance the fishery from October through April, but would have substantial adverse effect on the availability of water, and therefore fishery, between May and September. The detailed analysis is contained in Appendix O.

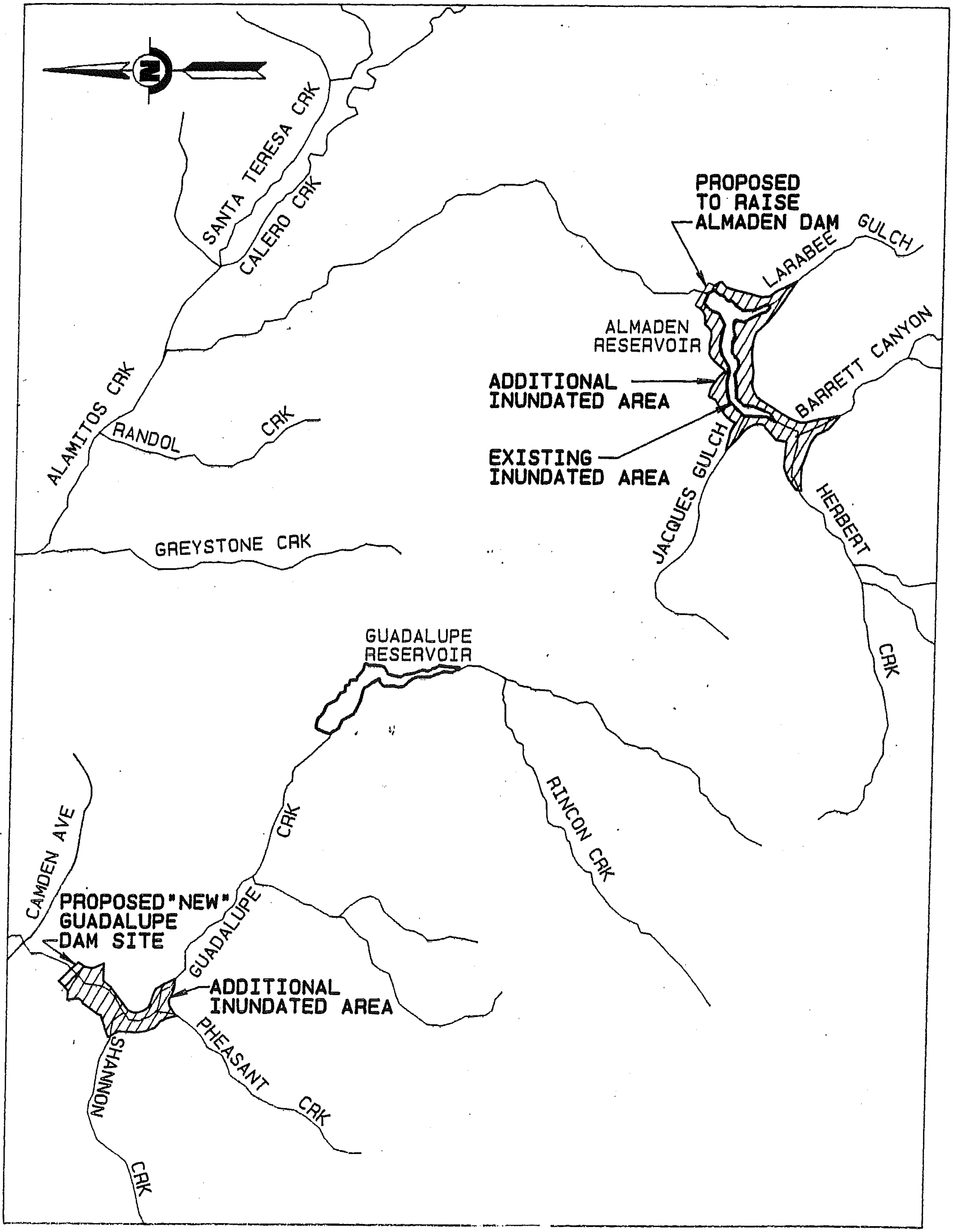

FIGURE 3.2
UPSTREAM STORAGE ALTERNATIVES

Off-Stream Storage

The concept of this alternative is to reduce the Guadalupe River 1 percent flow rate to an amount that can safely pass through the downstream channel by diverting flow into an off-stream storage pond. This would reduce the required downstream channel improvements. This alternative was investigated in detail in a 1977 report prepared by District staff (13). The objective of the plan was to eliminate the channel modifications measures that were proposed for Reaches 9-10A (Willow Glen Way to Canoas Creek).

The technical feasibility of this proposal is highly risky. The 1977 report showed that this plan must be designed to a very specific hydrograph peak, shape, and volume. Any deviation and the plan becomes dramatically ineffective. To improve the reliability of this plan with 20 percent contingency in peak flow, the storage volume required would be 6,250 AF.

The existing groundwater recharge ponds and adjacent area north of Blossom Hill Road was the only site large enough and at the necessary location to be considered. The total storage volume of the existing ponds is only about 650 AF. An additional 332 acres of land would need to be purchased and excavated approximately 17 feet deep to provide the additional 5,600 AF of storage. Some of the land has since been developed with condominiums and freeways, and therefore, no longer available.

Hydrologic constraints would require that the storage ponds be kept completely dry. This would be difficult due to variable groundwater levels, infiltration from adjacent recharge facilities, and other operational limitations. Once used to store floodwater, the ponds could not continue to operate as groundwater recharge facilities because the silt and debris would clog infiltration.

This alternative would still require some channel modifications in Reaches 6-8 to provide 1 percent capacity which would cost about $55 million.

This alternative is considered infeasible and impracticable because of high cost ($600 million—over $400 million is real estate cost). With the alternative, creek environment in Reaches 9 to 12 will continue to degrade. Implementing a future park and trail master plan will be difficult. Fish passage for upstream migration will not be restored.

Channel Modifications

This alternative proposed to modify existing stream channel to contain the 1 percent flood.

This alternative is considered feasible and practicable because it achieves most of the District's goals and objectives. The unavoidable impacts to riparian vegetation and wetland will be mitigated on site. This alternative will improve maintenance access and require less frequent maintenance in the natural channel. It will also reestablish a continuous riparian corridor along the creek and fish passage to upstream habitat. Implementing future trail master plan will be possible.

The following presents the descriptions of the different feasible channel modification alternatives investigated for each reach of the channel within the study area. These descriptions include a discussion of the different impacts of the alternatives and their estimated costs. The conservative

3:1 mitigation ratio for riparian vegetation impacts is for estimating and comparison purposes only.

Reach A: Highway 101 to Interstate 880

The river is straight and wide through this reach with side slopes varying from 1:1 to 2:1. The Airport Parkway and Airport Island bridges cross the river.

This reach has the capacity to contain the 1 percent flow but does not have adequate freeboard. There are some erosion problems in this reach.

The selected flood control alternative in this reach is compatible with the improvements proposed by Caltrans for Highway 87. In the interim, before Highway 87 is constructed, levees up to 3 feet high will be constructed on both banks.

The current plan for the Guadalupe Expressway Corridor improvements (Highway 87) is to raise the existing road between 4 feet to 18 feet and to excavate the east bank of the river between Airport Boulevard to I-880. The excavation would create a depressed bench up to 110 feet wide which would be used for maintenance access and for mitigation plantings for the highway project.

The only alternative considered in this reach is to lower the existing maintenance road on the west bank at Highway 101, Airport Island bridge, Airport Parkway bridge, and future Skyport Drive bridge to provide underpasses for a continuous maintenance road. The bank would be protected with stepped gabions at a 1:1 slope at each underpass. Ramps would also be constructed intermittently to provide maintenance access to channel bottom.

The estimated cost of this alternative is $1.3 million.

Reach 6: Interstate 280 to SPRR

The river is generally straight in this reach and the banks have side slopes varying from 1:1 to 2:1. High velocities that occur in this reach have caused bank erosion under Interstate 280 and degradation of the channel bottom throughout the reach. Rock-riprap was placed on the east bank from West Virginia Street bridge to 600 feet upstream to repair erosion that had occurred in 1982.

This reach is well under 1 percent capacity. The river from Interstate 280 to West Virginia Street would flood about every 3 years, and from West Virginia Street upstream the river would flood about every 5 years.

The West Virginia Street bridge crosses this reach. It was built about 1930 and is a reinforced concrete bridge with a center square-nosed pier. It does not have the capacity to convey the 1 percent flow.

Reach 6: Alternatives Considered

The feasible alternatives for this reach are the widened channel and the bypass channel (see Figure 3.3 and Table 3.7). The widened or bypass channel would be constructed to the east due to right of way limitation from the Guadalupe Expressway Corridor (Highway 87).

SANTA CLARA COUNTY FLOOD CONTROL
AND
WATER DISTRICT

SUMMARY OF DAM AND RESERVOIR INVESTIGATIONS
IN
SANTA CLARA COUNTY

by

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Office Engineer

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CALERO DAM AND RESERVOIR ENLARGEMENT PROJECT

The name of this project is "Calero Reservoir Enlargement Project". The engineering and economic feasibility study was done by Tudor Engineers for the District in December 1967. A separate geological and engineering investigation (first phase) was completed by Soil Mechanics and Foundation Engineers in February 1966. These investigations were of the detail needed for economic justification.

The purpose of this study was to establish the economic and engineering feasibility of enlarging Calero Dam and Reservoir for water supply, recreation and fish enhancement purposes. As a water supply facility, this enlarged reservoir would store additional water diverted from Almaden Reservoir, provide storage space for an emergency water supply and be operated as a terminal storage facility for the District's In-County Water Distribution System. In addition, this project would provide for a park facility around Calero Reservoir and support fish life. The expected increase in flood control benefits was considered to be very small and this purpose was not included.

In 1967 the engineering economic feasibility report was submitted to the State Department of Water Resources in support of the District's application for Davis-Grunsky construction grants.

The project is located on Calero Creek at the site of the existing dam and reservoir. This facility is within the City of San Jose near the intersection of Bailey and McKean Roads.

The project consists of the raising of main and auxiliary dams to elevation 520 feet (earthfill on the downstream face of main dam and earthfill on both faces of the auxiliary dam), a concrete lined uncontrolled spillway utilizing the existing spillway chute, a gated outlet works and extension of the existing outlet, an alteration of the downstream reach of the Almaden Calero Canal, the relocation of McKean Road and the development of a park facility around the reservoir.

The following criteria was established:

Hydrology

Drainage area - 6.9 square miles
Average annual runoff - Calero Drainage 2340 A.F.
Average annual diversion from Almaden Reservoir - 10,930 A.F.
100 year flood peak (Inflow) - 2710 cfs.
Maximum probable flood - 8690 cfs. (12 hour storm).

Dam and Reservoir

Type of Construction - earthfill section with inclined drain on the downstream face. Enlargement of main dam is on the downstream face (random fill with impervious cutoff) and for auxiliary dam the impervious fill is on the upstream face and the random fill is on the downstream face.

Top of Dam elevation - 520 feet (existing 490 feet)
Dam Height - 120 feet (existing 90 feet)
Crest length - 1050 feet
Crest Width - 20'
Side Slopes - Main dam 3:1
    Auxiliary dam upstream face 3:1, downstream face 2½:1

Reservoir Capacity -
    Total Capacity 24,000 A.F. (W.S. elev. 513 feet)-existing storage capacity - 10,750 A.F.
    Active Capacity 23,000 A.F. (W.S. elev. 512 feet)-existing storage capacity - 10,750 A.F.
    Normal Min Pool 6,000 A.F. (W.S. elev. 470 feet)

Reservoir Area at Maximum Storage 593 acres (existing 358 acres)

Outlet Works - Existing - gated 36 inch conduit
    Proposed - gated intake tower with 96 inch conduit.

Almaden-Calero Canal Extension - 66 inch pipe and siphon .
    Capacity - 120 cfs.

Spillway - Concrete lined ogee spillway at the location of the existing facility and tied to the existing chute.

Capacity - 2000 cfs
Crest length - 55 feet

Fill Quantity - Main Dam 1,444,000 Cubic Yards.

Auxiliary Dam

Crest Elevation - 520 feet
Crest Width - 20 feet
Crest Length - 2200 feet (existing 1600 feet)

Land Requirement

Reservoir Project - 494 acres
Total - 971 acres
Existing - 493 acres

The Calero Recreation area includes facilities for camping, picknicking, boating, swimming and fishing. These facilities include roads, parking area, boat ramps, fishing piers and a water supply and sanitary system. The following criteria was established:

Annual Visitation - 1,091,000 people
Picnic Units - 467
Camping Units - 140
Area - 176 acres
Boat Ramps - 4
Fishing Piers - 5
Comfort Stations - 24

The geologic and engineering feasibility investigation (stage 1) defined the geologic character of the area and engineering feasibility of raising Calero Dam. A major portion of the work was directed to defining the geologic character and engineering stability of the right abutment ridge of the main dam. This also included a preliminary dam modification design. This study concluded that geologic conditions are acceptable, seismic potential is moderate, the main and auxiliary dams can be raised economically, an impervious blanket is required at the right abutment, a buttress berm should be considered to increase the stability of the right abutment knob and the spillway can be raised at the present location.

The multipurpose project costs, construction and O.M.&R., based on 1970 prices are summarized below:

Construction Costs	
Reservoir Project	$6,321,000
Land Acquisition	4,746,000
Recreation Facilities	3,617,000
Water Supply and Sanitary Facilities	2,229,000
Sub Total	$16,913,000
O.M.&R. Costs	
Reservoir Project	342,000
Recreation Facilities	4,387,000
Replacement Facilities	3,225,000
Sub Total	$7,954,000
Total	$24,867,000

Net economic benefits for each of the purposes are summarized below:

Purpose	Benefits
Water Supply	$ 4,411,000
Recreation	18,338,000
Fish Enhancement	4,392,000

The project costs were allocated to the separate purposes based on the Separable Costs Remaining Benefits method. The cost allocated to water supply was $3,932,000 (15.8%) to recreation was $17,208,000 (69.2%) and to fish enhancement was $2,546,000 (15.0%). The benefit/cost ratio for the multipurpose project was 1.09.

Financing: In an agreement between the District and the County, the District will be responsible for the construction costs allocated to water supply and the related O.M.&R. costs and the County will be responsible for the construction costs allocated to fish enhancement and recreation and related O.M.&R. costs. A portion of the construction costs allocated to recreation and fish enhancement may be financed through State assistance in the form of grants made possible under the Davis-Grunsky Act. The requested grant for recreation is $5,534,000 and for fish enhancement is $1,265,000. In addition, a grant to construct water supply and sanitary facilities is available and the requested amount was $1,700,000. Considering these grants, the construction costs to be financed by the District are $2,278,000 and by the County are $6,135,000. Considering O.M.&R. costs, the total annual cost to the District is $167,000 and to the County is $872,000.

Conclusions: The multipurpose project as amended is economically feasible and should be constructed. Construction of the Reservoir Project should be completed by 1975 along with the East Side Recreation area. The remaining recreation areas would be completed by 1982, a 10-year development period. Considering a grant from the State in the amount of $6,800,000 adequate resources are available to construct and operate the facilities.

DAM AND RESERVOIR ON GUADALUPE CREEK

The name of this project is "New Guadalupe Dam and Reservoir". This study was done by the Santa Clara County Flood Control and Water District in 1968. A revision to this study was completed in May 1970. The engineering and geologic investigation was preliminary in scope. The purpose was to investigate the economics of constructing a second dam on Guadalupe Creek for water conservation, flood control and some measure of recreation. The development of hydro power was not considered.

The dam site is located on the main branch of Guadalupe Creek at Hicks Road downstream from the existing dam.

The project consists of an earthfill dam with a rock facing, concrete lined ogee crest spillway, gated outlet works and the relocation of Hicks Road.

Operation Criteria - 1968 Study

The objective of this study was to provide an optional measure of water conservation and flow regulation, together with flood control and a minimal provision for recreation. Therefore, water conservation was considered to be the primary purpose and flood control was introduced as a requirement for additional storage. A minimum pool was adopted for recreation but this purpose was not considered in the operation of the reservoir.

The combined operation of the existing and proposed reservoirs was considered and the optimum storage for conservation in the new reservoir was established based on operation-simulation computer runs using the Corps of Engineer's program "Reservoir Systems Analysis Conservation for 25 years of routing with 3 control points. The releases for water supply were assumed to be constant at 13 cfs (close to the average natural yield of the stream which is 12 cfs) with no seasonal variation. Computer runs based on conservation pools ranging from 12,080 AF to 32,000 AF indicated that the optimal storage capacity was 23,000 AF. Considering other economic factors a slightly smaller water conservation pool of 19,300 AF was suggested.

Provisions for flood control were included as a minimum storage above the conservation pool need to control the 100 year flood. Releases from the new dam combined with the natural runoff to the confluence with Alamitos Creek were limited to not more than 1600 cfs. Two alternatives were considered 1) Store the 34 hour-100 year flood volume estimated to be 7000 AF and 2) store the same flood volume as in alternative 1 to limit the downstream flow to 1600 cfs. - estimated volume 3000 AF.

Recreation was not considered in the operation criteria but a minimum pool of 2000 AF was adopted for this purpose and for silt storage.

Hydrology

a. Level of flood protection - 100 year flood (34 hour volume = 7000 AF and peak inflow = 12,680 cfs) way To high

b. Maximum probable flood - outflow
   with 3000 AF of flood control storage 9,350 cfs
   with 7000 AF of flood control storage 1,500 cfs

c. Storage requirement to limit 100 year downstream flood flows to 1600 cfs is about 3000 AF

d. Annual yield (19,300 AF of storage) is 9040 AF

Dam and Reservoir Design Criteria:

1. Type of Dam - Zoned earth embankment with 3 to 1 side slopes. (Impervious core with rock fill).

2. Reservoir capacity - 24,300 AF (19,300 AF for conservation 3000 AF for flood control and 2000 AF as minimum pool).

3. Outlet works capacity 1600 cfs at minimum pool elevation (66" diameter pipe).

4. Spillway size and capacity

   a. 200 feet long with a capacity of 9350 cfs at a head of 5.6 feet (3000 AF storage for flood control)

   b. 40 feet long with a capacity of 1500 cfs at a head of 4.75 feet (7000 AF storage for flood control).

5. Height of dam above stream bed - 199 feet (Elevation 524.5 feet).

6. Top of flood control pool - 524.5 feet.

7. Fill quantity - 7,100,000 cubic yards.

Capital costs for the dam and reservoir (without appurtenances) were computed based on 3 alternative water conservation pools and 2 flood control pools. These costs were for comparative purposes and are summarized below based on 1968 prices.

Storage Capacity (Total) AF	Capacity for Water Cons. AF	Capacity for Flood Control AF	Total Cost $\$ \times 10^6$	Allocated Cost		
				Rec. $\$ \times 10^6$	Water $\$ \times 10^6$	F.C. $\$ \times 10^6$
31,000	22,100	7000	14.1	.9	11.4	2.1
28,300	19,300	7000	13.1	.9	10.1	2.1
21,080	12,080	7000	10.2	.8	7.5	1.9
27,100	22,100	3000	12.6	.8	11.4	.4
24,800	19,300	3000	11.4	.8	10.1	.5
17,080	12,080	3000	9.8	.8	7.5	1.5

Note recreation pool is 2000 AF

Total costs for 3 alternative reservoirs with 3000 AF for flood control are summarized below:

Reservoir Capacity AF	Dam & Res. $\$ \times 10^6$	Spillway & Outlet $\$ \times 10^6$	Total Cost $\$ \times 10^6$	Allocated Cost		
				Res. $\$ \times 10^6$	Water $\$ \times 10^6$	F.C. $\$ \times 10^6$
17080	9.8	0.7	10.5	0.9	8.0	1.6
24300	11.4	0.8	12.2	0.9	10.8	.5
27100	12.6	0.9	13.5	0.9	12.2	.4

Annual costs are $456,000 (17,080 AF storage), $615,000 (24,300 AF storage) and $695,000 (27,100 AF storage) and the costs per acre foot are $53.10, $68.00 and $76.50 respectively.

May 1970 Restudy of the Proposed Reservoir

In the May 1970 revision the effects of 1) the enlarged Calero Reservoir, 2) the proposed Almaden-Calero Reservoir operation and 3) the proposed new dam on Guadalupe Creek were considered in a restudy of the hydrology and operation of the Guadalupe River system. The new Guadalupe River Reservoir was considered as a 1) multipurpose reservoir for flood control, water supply and recreation, 2) as a flood control and recreation facility and 3) as a single purpose flood control facility.

Changes in the downstream flood control flows caused by the operation of the Almaden and Calero Reservoirs appear to be very small and it was assumed that the 1968 hydrology and operating criteria for the proposed Guadalupe Reservoir was reasonable.

Costs based on 1970prices for the single purpose reservoir and 2 multipurpose reservoirs are summarized below. The water conservation storage pool was set at 19,300 AF and 3000 AF was assigned to flood control (minimum pool 2000 AF).

Project	Total Cost $\$x10^6$	Allocated Cost Res. $\$x10^6$	Water $\$/10^6$	F.C. $\$/10^6$	Annual Costs 5% int. $\$/10^3$	6% int. $\$/10^3$	8% int. $\$/10^3$
Multipurpose Proj.(3 pur.)	13.7	1.5	10.2	2.0	840	951	1,184
Multipurpose Proj.(2 pur.)	4.3	-	1.7	2.6			
Single Pur. Project	3.2			3.2			

For the multipurpose project the cost of water conserved ranges from \$69/AF to \$97/AF.

Conclusions: The cost per acre foot for conserved water delivered from the new Guadalupe Reservoir could not compete with the cost of imported water (\$68/AF vs. \$37.40/AF for imported water) and the average annual yield falls somewhat short of the proposed objectives. Since it was difficult to determine flood control benefits, it was felt that the cost of a single purpose flood control facility should be compared with the cost of downstream improvements. The cost of downstream improvements was estimated to be \$1,900,000 which is much lower than the single purpose cost of \$3,200,000 or the multipurpose allocated cost of \$2,550,000.

The operation of Calero and Almaden Reservoirs as proposed in the Calero Reservoir Report of 1967 has little affect on downstream flood peaks and any changes would result from the enlargement of Calero Reservoir.

Comments: It appears that this second reservoir on Guadalupe Creek is not economically justified. It is cheaper to purchase imported water and construct channel improvements downstream. Based on these large cost differences, it may be difficult to justify the water supply and flood control purposes by refining the cost allocation.

DAM AND RESERVOIR SITE ON GUADALUPE CREEK

The name of this project is "Second Dam and Reservoir on Guadalupe Creek". The engineering feasibility study was made by the former Santa Clara Valley Water Conservation District and the work was completed in 1956. The engineering and geologic analysis was preliminary in scope. The purpose of the investigation was to establish the engineering and economic feasibility of constructing a second dam and reservoir on Guadalupe Creek for the conservation of local runoff for ground water recharge and flood control.

This project is located on Guadalupe Creek near the mouth of Guadalupe Canyon about 3/4 miles downstream from Shannon Road. The drainage below the existing dam is 7.44 square miles and the total drainage area is 13.44 square miles.

The project consists of an earth filled dam on Guadalupe Creek and includes a concrete lined chute type spillway, gated outlet works and diversion structures for offstream percolation facilities to be constructed in the area along Guadalupe Creek between the hills and Los Alamitos Creek.

Dam and Reservoir Design Criteria:

1. Type of dam - earth filled with rock facing.
2. Height of dam above stream bed - 110 feet
3. Crest length of dam - 2375 feet (Elevation 395 feet).
4. Fill Quantity - 1,500,000 Cubic Yards
5. Spillway design capacity - 10,000 cfs.
6. Maximum water surface elevation - 380 feet (freeboard 15 feet).
7. Reservoir Capacity - 5500 Acre Feet.

The geologic investigation was preliminary and it was determined that two faults and sheared rock zones underlie portions of the reservoir. Leakage through the bottom of the reservoir into the old Guadalupe mines could be a problem. In addition, a large amount of stripping is required to expose acceptable foundation material and construction materials are available at the site.

Local runoff that is not stored in the existing Guadalupe Reservoir and runoff originating below the existing dam is stored in the lower reservoir and released into the main creek bed for ground water recharge. This flow is diverted downstream to offstream percolation ponds. By providing a storage reservation in both reservoirs during the flood season (no valves shown), a large measure of flood protection to the North San Jose area could be provided. No estimate of the benefits was made and no criteria for flood control was given.

The estimated total construction cost based on 1955 prices was estimated to be $2,524,000. Based on the ENR cost index the estimated present cost (1970 prices) is about $4,600,000. The cost per acre foot based on local financing (1955 costs) was estimated to be $21 and with Federal assistance was about $17.

Recommendations: The project was considered feasible and recommended for construction as a flood control and conservation facility.

Comments: Recreation has not been considered as a purpose, and no attempt was made to establish benefits and allocate costs to the separate purposes. Considering the potential recreation and flood control benefits associated with this project, it appears that this project should be looked at in greater detail.

SANTA CLARA VALLEY WATER DISTRICT

LEXINGTON RESERVOIR
ENLARGEMENT FEASIBILITY STUDY

by

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Under the Direction of

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February 1974

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SYNOPSIS

A preliminary economic feasibility study of enlarging Lexington Reservoir from the present 20,300 acre-feet capacity to a capacity of 40,600 acre-feet was conducted. The total value of the project benefits consisting of additional water supply or flood control, recreation and emergency water supply was compared to the total cost of the project consisting of the costs for raising the dam by an additional 40 feet, relocation of Highway 17, compensations for San Jose Water Works facilities affected by the reservoir enlargement and acquisition of right of way. The benefit to cost ratio was found to be 0.96, under the assumption that for the given enlargement of the project size, a proportionate increase in recreational facilities can be made that would normally accommodate an additional 300,000 recreational visitors annually. However, if the enlarged reservoir were to have more extensive recreational use with facilities expanded for 500,000 or even 1,000,000 visitors annually the benefit to cost ratio increased to 1.06 and 1.24. Thus, recreational use was found to be critical in the feasibility study.

Future studies may consider in more detail the recreational costs and benefits, reservoir operations for flood control and conservation, geologic feasibility, feasibility of enlarging the reservoir to a lesser extent and the feasibility of operating the reservoir as a multi-purpose facility.

SUMMARY AND CONCLUSIONS

A preliminary study on the feasibility of enlarging Lexington Reservoir as a conservation, flood control and recreation facility was conducted. In the study, it was considered that Lexington Dam would be raised by 40 feet, thereby doubling the reservoir storage capacity from its present value of 20,300 acre-feet (existing spillway crest elevation, 650 feet) to a maximum of 40,600 acre-feet (spillway crest elevation, 690 feet).

For the present study, only the economic feasibility of enlarging Lexington was studied. Geologic feasibility would be studied if the project proved economically sound. The hydrologic investigations were limited to yield determinations and peak flood flow reductions for determining preliminary benefits.

The major cost items include the costs of construction of the dam, relocation of Highway 17, the acquisition of rights of way and compensation to San Jose Water Works for their water supply facilities that would be affected. These facilities are (i) the Montevina Filter Plant located on the west side of the reservoir, (ii) some of the intake facilities located upstream from the dam, and (iii) raw and treated water pipelines. Several alternative plans for relocating and/or protecting these facilities were studied, and the least-costly alternative plan was used in the benefit-cost analysis.

The benefits consisted of (a) varying amounts of additional water conservation and flood control protection, (b) additional recreation and (c) emergency water supply storage. The conservation and flood control benefits were dependent on various schemes of reservoir operation. The value of additional recreational benefits was directly related to the additional visitation potential that would result by the expanded recreational facilities. Consideration was also given to using an enlarged Lexington Reservoir to store raw water to be used in an emergency by the Rinconada and Penitencia Water Treatment Plants.

The summary of annual costs and benefits and benefit-cost ratios corresponding to different schemes of reservoir operation are presented on Table 6. As can be seen, none of the combinations of conservation, flood control, recreation and emergency storage, yielded an overall benefit-cost ratio in excess of unity. Inclusion of recreation improved the benefit to cost ratio although it was still slightly below unity.

The cost and benefit values due to recreation were based on an estimated 300,000 additional visitors annually to the enlarged reservoir.

TABLE 6. BENEFIT-COST RATIOS FOR THE PROPOSED LEXINGTON ENLARGEMENT

	Annual Costs in Millions of Dollars				Annual Benefits in Millions of Dollars					Benefit-Cost Ratio	
Type of the Reservoir Operation	Water Supply and Flood Control	Recreation	Emergency Storage	Total	Water Supply	Recreation	Flood Control	Emergency Storage	Total	Without Emergency Storage	With Emergency Storage
Conservation only. Reservoir filled during winter flows when possible.	1.33	0	0.24	1.57	0.55	0	0.05	0.59	1.19	0.45	0.76
Conservation and recreation; no flood control as reservoir is maintained nearly at full enlarged capacity.	1.33	0.3	0.24	1.87	0.55	0.60	0.05	0.59	1.79	0.74	0.96
Recreation and a combination of conservation and flood control with enlarged reservoir maintained at 3/4 full.	1.33	0.3	0.24	1.87	0.30	0.60	0.16	0.59	1.65	0.65	0.88
Flood control; no additional conservation as enlarged reservoir is maintained at ½ full capacity. Recreation included.	1.33	0.3	0.24	1.87	0	0.60	0.38	0.59	1.57	0.60	0.84

Note: No allocation of costs of reservoir enlargement have been made between water supply, recreation, flood control, or emergency storage; therefore no separate evaluation as to the Benefit-cost ratios of the separable purposes can be made on the basis of information presented in this table.

When annual visitations of 500,000 and 1,000,000 were considered in the analysis, the benefit cost ratio exceeded one for both levels of visitation. These visitation figures may be achieved if the enlarged reservoir were constructed but such a magnitude of use may not be desirable. The benefit and cost values corresponding to these visitations are presented below.

Annual No. of Recre. Visitor/ Days	Annual Costs ($Mil.)			Annual Benefits ($Mil.)			Overall Benefit/ Cost Ratio
	Recre.	Other Uses	Total Uses	Recre.	Other Uses	Total Uses	
500,000	0.50	1.56	2.06	1.00	1.18	2.18	1.06
1,000,000	1.00	1.56	2.56	2.00	1.18	3.18	1.24

Thus, the project appears to be economically feasible only when a visitation for recreation use in excess of 500,000 per year is accommodated.

Future studies on Lexington enlargement should include the following:

(i) More detailed analysis on recreational costs and benefits (considering that in the present study these were evaluated approximately, being based on escalated unit values from the Calero Study).

(ii) Reservoir operation studies to determine more precisely the relative proportions of conservation and flood control benefits.

(iii) Feasibility of enlarging the reservoir a lesser amount.

(iv) Geologic feasibility of raising the dam.

(v) Feasibility of operating the reservoir as a multi-purpose facility.

Preston

GUADALUPE RIVER

EVALUATION OF OFFSTREAM
STORAGE ALTERNATIVE

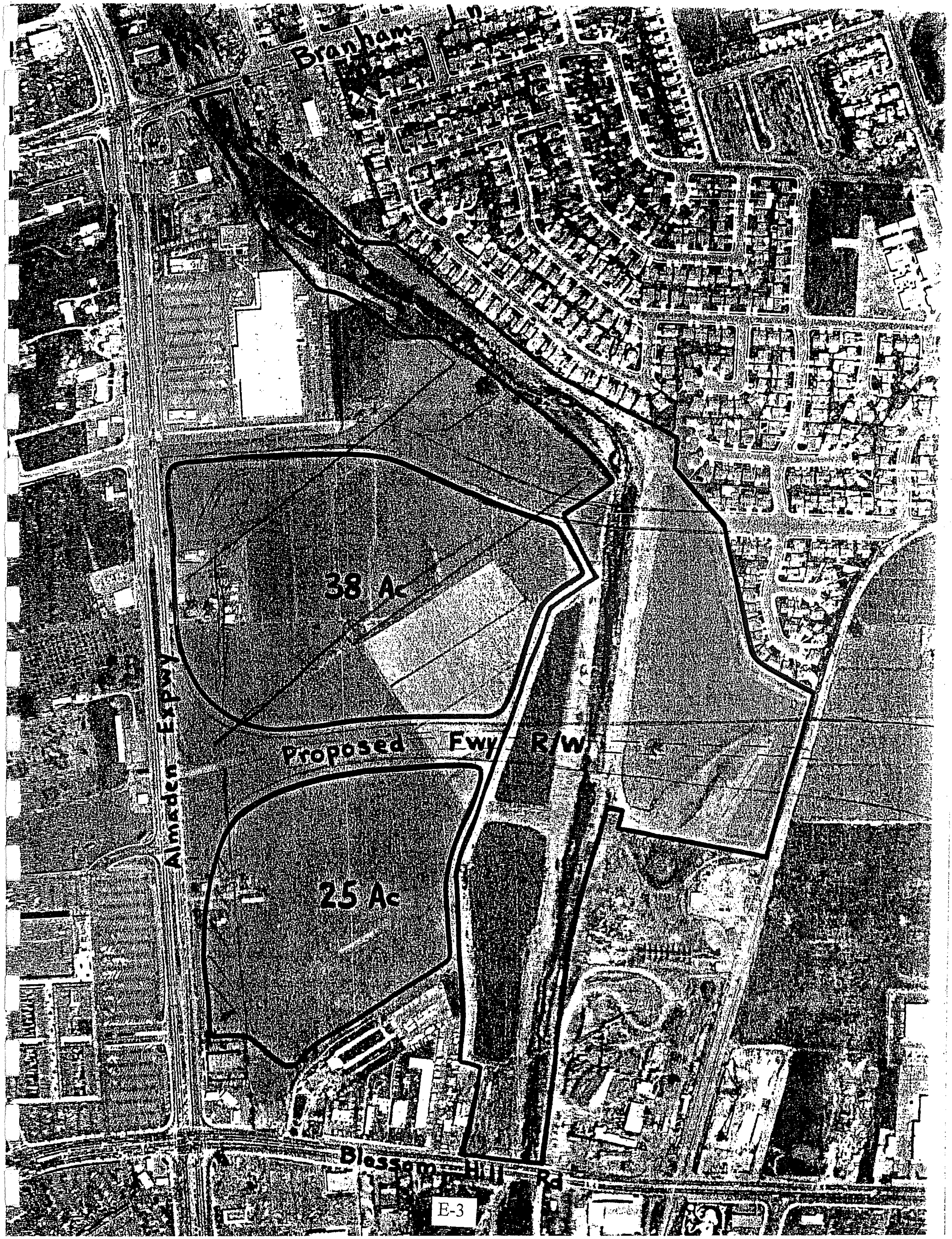
(ALT. #6)

April, 1977

Randy Talley, P.E.

OBJECTIVE

The objective of this study is to evaluate the feasibility of using offstream storage as a flood control facility to eliminate or reduce the need for extensive channel improvements on the Guadalupe River. The singular objective was to eliminate the need for floodwalls and selective clearing in Reach IX, Willow Glen Way to Curtner Avenue.


Branham Ln
38 Ac
Almaden Expwy
Proposed Fwy R/W
25 Ac
Blossom Hill Rd

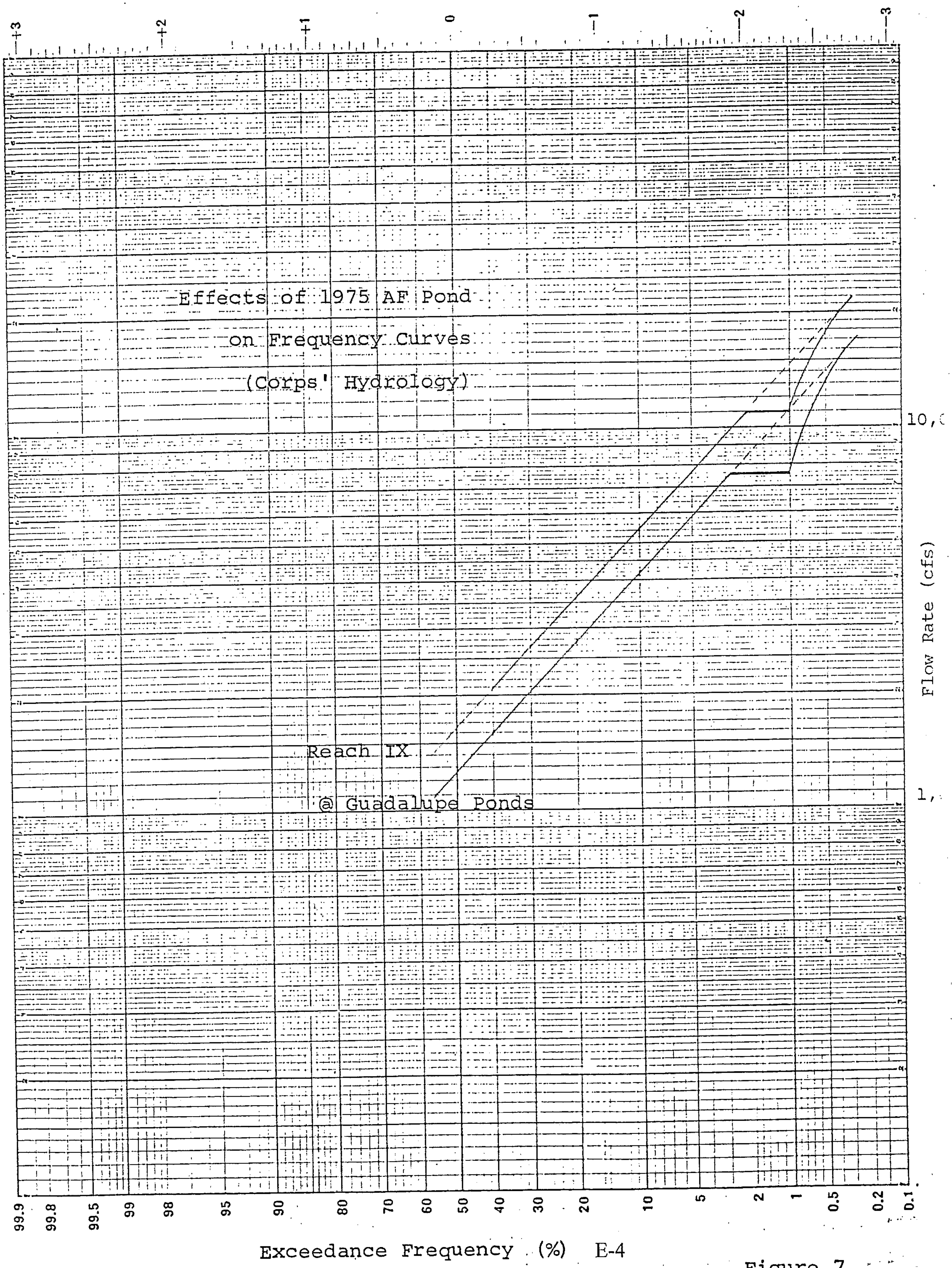

+3
+2
+1
0
-1
-2
-3
Effects of 1975 AF Pond
on Frequency Curves
(Corps' Hydrology)
Reach IX
@ Guadalupe Ponds
10,
1,
Flow Rate (cfs)
99.9
99.8
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
Exceedance Frequency (%)

Figure 7

CONCLUSION AND RECOMMENDATIONS

It is technically possible to construct a combination catchment pond-diversion structure which will trap the peak of the Corps' 1% hydrograph; thus reducing the flow rate downstream by 3,600 cfs.

The possibility of quarrying the gravels on the proposed site might make the pond construction economically feasible.

This is the only study which looked at the possibilities of multi-purpose uses and savings in cost of downstream channel improvements. The Corps should be asked to reevaluate all other alternatives in this same manner to determine if there might not be an even better proposal. All Corps' analysis thus far have been for single-purpose facilities.

If the decision is made to go with the enlarged pond alternative, immediate action is needed to purchase the entire 63 acres. Deals on these parcels are now pending.

This report should be circulated in-house and to the Corps for review. If approved, the proposal should be incorporated into a supplemental EQ plan.

TA710.3C38 [illegible]

STUDY OF FACTORS INFLUENCING ERODIBILITY OF SOILS IN UNLINED EARTH CHANNELS IN THE SANTA CLARA VALLEY

PREPARED
FOR

SANTA CLARA COUNTY FLOOD CONTROL & WATER DISTRICT
516 EAST MARTHA STREET
SAN JOSE, CALIFORNIA

BY

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(408) 297-9585

March 5, 1971
Project JS 11899

Santa Clara County
Flood Control & Water District
516 East Martha Street
San Jose, California

Gentlemen:

We have made a study of factors which influence the erodibility of soils forming the periphery of unlined earth channels in the Santa Clara Valley. This work was performed pursuant to Work Order Number 81206 issued by the Santa Clara County Flood Control & Water District.

Our purpose was to study the erosion existing in unlined flood-control channels and the associated soil conditions. Based on the results of this study, recommendations have been formulated for a design procedure to predict and limit erosion of unlined-channel banks. The accompanying report contains our conclusions and recommendations as well as the results of our field investigation and laboratory tests.

During this study George M. Sicular, Professor of Civil Engineering, San Jose State College, served as a consultant. He contributed specifically toward evaluation of hydraulic factors and with critical literature appraisals.

Very truly yours,

William A. Bischoff

William A. Bischoff
Project Engineer

Reviewed by

Robert S. Wright

Robert S. Wright
Senior Project Engineer

WAB:bb

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ERODIBILITY OF SOILS IN UNLINED EARTH CHANNELS IN THE SANTA CLARA VALLEY

INTRODUCTION

Unlined flood control channels in the Santa Clara Valley have been continually eroding. Channel deterioration of this type has presented problems of right-of-way encroachment and loss of hydraulic characteristics. This report summarizes a study made of the soil erosion phenomena. The study was limited in scope to an investigation of the erodibility of soils in straight unlined channels.

To determine the state of the art of stable channel design, a brief literature review was made. In addition, existing design techniques used by the District and those used by other government agencies were investigated. These items are included in appendixes to this report.

Visual examinations were made of several selected channels within the District's jurisdiction. Soils exposed in the channel banks were sampled, classified and tested in our laboratory. Soil types and properties were correlated with existing channel condition and with estimates of maximum water velocities.

Several factors influencing deterioration of unlined channels were apparent from this study. Firstly, maximum water velocities occurring during peak flows are creating forces on the channel surfaces which exceed the resistance of the channel soil materials. Secondly, slope instability was observed as a major cause of channel deterioration. Piping through channel banks and soil shrinkage were observed to contribute to slope instability.

Recommendations for future channel design are contained in the following sections along with suggested areas for future study and research.

CHANNEL INSPECTIONS

Six channels within the Santa Clara Valley were inspected and sampled. Erosion was noted in all channels to varying degrees. Inspections were limited to channel banks as bed-load covered all channel bottoms. In this report, the degree of erosion is defined as follows:

1) Negligible - Less than 3 in. of soil eroded from bank. Channel practically remaining in an "as-built" condition.
2) Slight - 3 to 12 in. of soil eroded from banks.
3) Moderate - 1 to 3 ft of soil eroded from banks.
4) Severe - More than 3 ft of soil eroded from banks.

Following are descriptions of each channel inspected. A detailed description of soil samples, laboratory test results, and estimated maximum flow quantities are contained in Table I and Table II.

Ross Creek (Improved in 1956)- Ross Creek was inspected between Camden Avenue and the Almaden Road as shown in Figure 1. Channel soil materials consist of dense sandy gravel deposits alternating with light brown silty clays along the channel length.

Overall erosion can be classified as slight to moderate. The majority of erosion has occurred within a depth of approximately 0 to 4 ft from the existing channel bottom. Erosion has occurred in uniform sheet-like sections triggering

Erosion of this channel is classified as slight and limited to the bottom 3 to 4 ft of channel. No major bank failures were noted. The presence of flowing water in the creek during inspection precluded detection of piping through channel banks. Typical channel condition is shown in Figure 9.

Golf Creek (Improved 1965) - Golf Creek was inspected between the Almaden Expressway and Culligan Boulevard as shown in Figure 6. Although no actual soil sampling was done, a qualitative evaluation of the soil conditions has been made. The materials exposed within this channel varied from silty clays to dense clayey sand-and-gravel deposits. Erosion, which has occurred to varying degrees, can generally be classified as moderate.

Some soil bank erosion has occurred and has precipitated landslides infringing upon private property. It is estimated that in the near future additional sliding will occur within the clay deposits causing property boundary problems. On the average, bank slopes have eroded to a 1 to 1 inclination. (See Figure 10).

CHANNEL DETERIORATION

Examinations of straight channel sections indicate deterioration is occurring to varying degrees in all channels. One cause of channel deterioration is erosion. A second cause of channel deterioration is bank instability. These items and other causes of deterioration are discussed in the following sections.

Erosion - Erosion is defined as the wearing away of soil by the shearing action of water moving over the channel surface. Shearing stresses created by flowing water are called tractive forces. Erosion caused by tractive forces is resisted differently in various soil types. Both the creation of tractive forces and soil resistance to tractive forces are discussed below. The tractive force concept appears to be the most valid approach to analysis of channel erosion. In the past, the District has used the allowable water velocity approach. Water velocities are also discussed in the following sections for comparison with channel erosion.

a) Tractive Forces & Water Velocities - Field observations indicate that excessive tractive forces and/or velocities have occurred in the channels inspected. This is apparent from the evidence of channel erosion. In an attempt to define the erosive forces, estimates of the maximum developed tractive forces and channel water velocities have been made from recorded maximum flows. These estimates are presented in Table II.

To make these calculations, it was necessary to assume an as-built channel cross section and bottom slope. Calculated tractive forces were also based on recorded maximum flow depths and soil grain sizes. Soil grain sizes, as determined by laboratory tests, are presented in Figures 16 through 27. Where soil grain sizes have not been determined, estimates have been made for calculation purposes.

Calculated tractive forces are based on a modification of Lane's/[16] technique as defined in Appendix A. Lane's equation yields low (calculated) tractive forces for fine grain soils. Recommendations are contained later in this report to impose a limit upon this equation and to provide realistic friction factors for these fine grain soils.

Based upon a modification of Lane's equation with new recommended limits, maximum tractive forces have been calculated and are summarized in Table II. For the channels studies (except Smith Creek) all calculated tractive forces exceed the allowable for stable channels. (Allowable tractive forces are discussed in the following section)

Calculated maximum channel velocities are also presented in Table II. Although the tractive force method of analysis provides a more direct procedure for evaluation of channel erosion, maximum channel velocities have been compared with "allowable velocities" to provide generalized empirical evaluations. Allowable channel velocities in Table II are those presented in Technical Release #25 prepared by the United States Soil Conservation Service./[23]

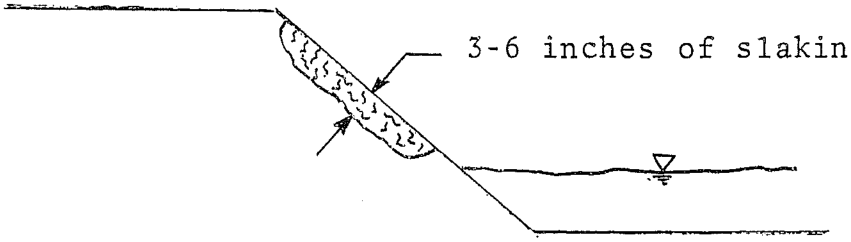
Excessive tractive forces and/or channel water velocities are created primarily by excessive channel bottom slopes. Design velocities of 8 ft per second used by the District in the past have been excessive and have developed excessive tractive forces. Siltation and bed-load have further increased channel gradients which increased the already excessive tractive forces. This effect was studied during the heavy flows of January 1969 in San Tomas Creek by Mr. Edwin Ferguson, Senior Design Engineer for the District. It was reported that materials were deposited below drop structures steepening the bottom slope to something near the original channel configuration. This in turn resulted in excessive tractive forces and consequent channel erosion.

b) Resistance to Erosion - Based on our observations, it is apparent that resistance to tractive forces is developed differently in cohesive and non-cohesive soils. In cohesionless soils, erosion or particle removal occurs when the tractive forces created by flowing water are sufficient to lift the particle and transport it downstream. Resistance to the erosion, therefore, is provided by particle weight and interparticle friction. Usually erosion of cohesionless soils occurs in the form of sheet-type soil removal in long uniform strips.

Cohesive soils provide resistance to erosion by virture of their cohesive strength. Interparticle bonding of the clay materials provides this strength. Erosion of clay materials occurs when the interparticle bond forces (cohesion) have been exceeded or weakened. The particles then become suspended in colloidal forms. Particles in over-consolidated clays at natural moisture contents normally are held in place by relatively high interparticle forces and consequently offer high resistance to tractive forces.

If dried, cohesive materials develop high strengths. Highly plastic materials form exceptionally high interparticle bonds when drying occurs. This drying-induced bond (cohesion) is not a permanent strength increase and rewetting of cohesive soil will destroy the interparticle bond resulting in a loss of strength. Where the bond is nearly destroyed a cohesionless suspension of fine-grained particles results and the "slaked" particles are particularly vulnerable to tractive forces and erosion.

Based on field observations, it is estimated that depths of 3 to 6 in. of slaking occur on channel sides during normal seasonal changes. This depth of soil is easily and quickly eroded from the channel bank by either high water flow or rainfall.

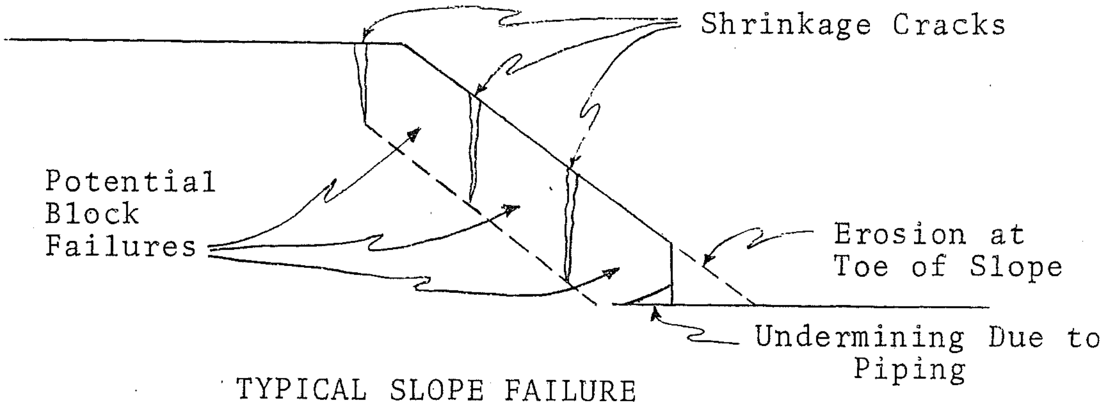


SLAKING

Slope Failure - Slope failures in clay soils appear to be the main cause of property encroachment in flood-control channels. These slope failures may be defined as large masses or blocks of soil which slide or fall from the banks into the channel bottom. Slope failures create a pockmark appearance on the channel sides, generally leaving near vertical side slopes.

The mechanism of slope failure is a progressive one in which the toe of the slope is first worn away by both erosion and/or piping. Piping usually occurs in fine-grained, cohesionless materials. Ground water flow from an adjacent area into the channel tends to loosen the soils through which it flows. At the toe of the channel slope, high seepage gradients are created. Because these high gradients saturate and loosen the soils, tractive forces created by flowing water easily erode the toe of the slope which in turn precipitates slope failures.

Tension cracks, caused by drying, create additional slope instability. When these cracks become filled with water or are extensive enough to lessen the strength of a slope below that necessary to hold the material, a slope failure occurs. The material then falls into the creek channel and is carried away. This type of slope failure is different from the usual circular arc failure assumed in channel design.



TYPICAL SLOPE FAILURE

Based on our inspections, it appears that slope failures create the major problems of channel deterioration. Slope failures remove the largest volume of material and are the primary cause of property encroachment.

Channel Design - Determination of the soil conditions along a proposed channel is an important part of design. Soil types are often variable along natural channel beds due to meandering of the stream and subsequent deposition of different materials. Recent stream deposits are usually loose and weak.

The District presently evaluates the soil conditions along a channel alignment from exploratory borings made on about 1000 foot centers. Laboratory tests are made on soil samples to determine grain-size distribution, plasticity index and shear strength.

Average soil conditions, as are being used across a given soil cross section, result in unconservative channel designs. For example, the soil cross section along a District designed, unlined channel improvement consisted of a stiff silty clay underlain by a low-plasticity silt at a depth of about 3/4 the channel depth. The silty clay, being the predominant material, was used for design considerations; however, the silt material, which has the lesser resistance to tractive forces and erosion, is located in the area subject to the major flow. In this same case example, a uniform soil condition was assumed for slope-stability calculations when a non-uniform and weaker actual condition existed. Greater attention should be given to details of the soil-profile.

Alteration of Flow Characteristics - Erosion was also observed due to channel irregularity (such as curvature, change in channel roughness, etc.) that alter the flow characteristics. Water passing over a change in channel roughness or channel irregularity causes a vorticity effect which creates flow into or away from the channel bank rather than parallel to it. This type of flow has a great eroding potential.

RECOMMENDATIONS

Design - An important part of design is the recognition of soil conditions along any proposed alignment. It is recommended that the existing exploration program of borings on 1000-foot centers be continued for minor projects. On major projects, where right-of-way consideration are more important, boring spacing should be decreased to 500-foot intervals. Where improved channel alignments follow existing natural channels, a complete soil and geological reconnaissance of existing banks should be made. Laboratory tests should be made to determine the Atterburg Limits and grain-size distribution. Shear-strength tests should also be made.

Based on the field investigation and laboratory tests, typical soil cross sections should be drawn for each interval investigated. The susceptibility of each soil to tractive forces and each soil position within the channel section should be studied. The same factors should be considered in slope stability calculations. Field inspection during construction should be made to verify the anticipated soil conditions and to detect any undesirable conditions which might be present.

It is recommended that a tractive-force design procedure be used for future channel improvements. A tractive-force analysis is one of successive trials. A trial channel section must first be designed from which the tractive forces can be calculated. These calculated forces should then be checked against the allowable tractive forces for the particular soil material. If the developed tractive forces exceed the allowable, a second, or possibly a third, trial section must be designed.

For a first estimate of channel design, it is recommended that an allowable velocity be selected from the charts and table on Figure 6-1 of the Soil Conservation Service Manual TR-25./[23] Actual developed tractive forces should then be calculated. For soils with a D_{75} particle size greater than 1/4 inch, developed tractive forces can be calculated by the technique recommended by Lane./[16] This technique is described under the "Literature Search" Appendix of this report. With Lane's technique, an equivalent Manning's "n" associated with soil-water friction is determined by the following equation:

$$n = \left(\frac{D_{75}}{39}\right)^{1/6}$$

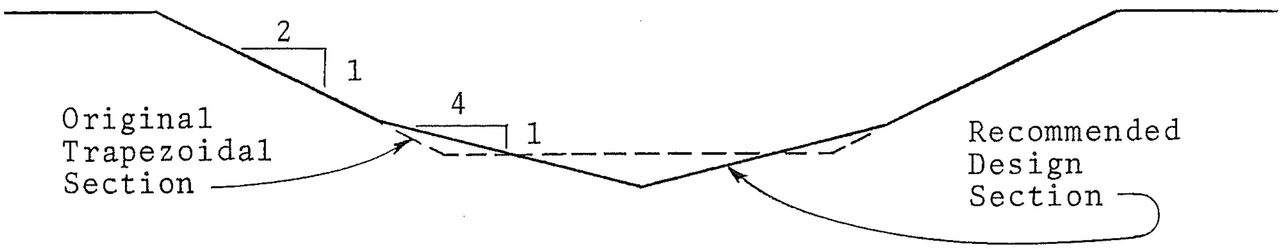
n = Equivalent Manning's "n"

D_{75} = Soil particle size of which 75 percent of soil sample is smaller.

Extrapolation of this relation to soils with D_{75} less than 1/4 inch produces unrealistically low values. For example, in channels without vegetation, the impedence to flow due to soil friction can be calculated to be as low as 30 percent of the total over all impedence. This relation is obviously unrealistic. As a limiting value, it is recommended that the equivalent Manning's "n" used to calculate developed tractive forces should not be less than 85 percent of the total Manning's "n" used for channel design. After the developed tractive forces have been calculated, the soil allowable tractive forces should be determined from Figure 15 of this report. This figure is based on a compilation of the work of Lane,/[16] Dunn,/[6] Smerdon & Beasley,/[26] and the Soil Conservation Service/[23] which has been adjusted based on the results of this study.

The values given for fine-grained soils should limit erosion provided that the clays are maintained at their natural moisture content. Clay banks subjected to cyclic wetting and drying will slake. Slaking depths of 3 to 6 in. per season should be expected unless some means is employed to prevent loss of natural moisture. Drying might be prevented by polymer sprays or other compounds. This is an area in need of future study.

It has also been noted that most erosion begins at the toe of slopes, eventually causing slope instability. An increase in channel longevity might be attained by changing the trapezoidal channel cross sectional shape to the configuration shown below:



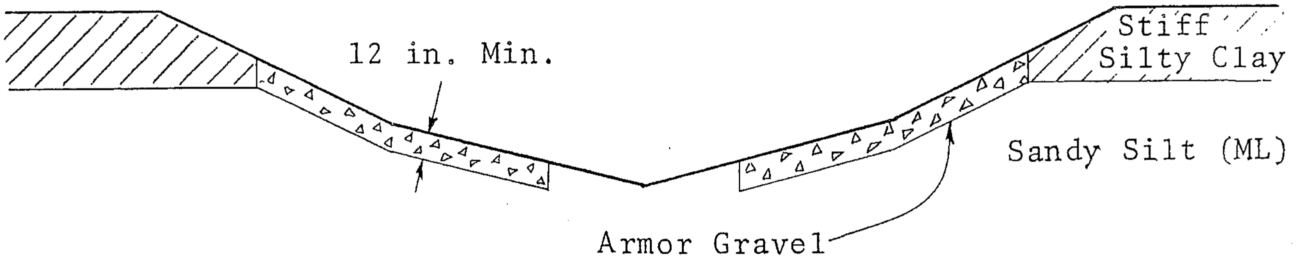
This configuration would confine low flows to the center of the channel and would require that a larger volume of material to be eroded before an unstable bank is created.

In areas where fine cohesionless soils are exposed at the toe of the channel bank, piping conditions could develop due to ground water seepage. Soils having the following material properties should be considered susceptible to piping:

$D_{50} < 1.0$ mm D_{50} = Particle size of which 50% of soils is finer.

Plasticity Index $<$ 15

The effects of piping could be minimized by excavating these fine-grained soils and replacing them with a pervious blanket of armor gravel and filter material as shown below.



The armor rock blanket should be a well-graded filter material having the following gradational restrictions:

$$5\text{mm} < D_{85} < 75\text{mm}$$

$$0.075\text{mm} < D_{15} < 1.1\text{mm}$$

This material should also have an allowable tractive force resistance sufficient to resist the developed tractive force at maximum flow. A minimum section of 12 in. of armor rock should be placed for protective purposes. This armor blanket is intended to stabilize fine-grained soils against piping and subsequent erosion and is not intended to be riprap. Riprap is usually well graded rock materials having a D_{15} size of 3 in. Where riprap is needed to resist scour such as on bends it should be placed on a bedding layer of sandy gravel filter material.

The gradational limits of the armor gravel are quite broad. It is anticipated that a material meeting these requirements might be found within most channel improvement areas making use of such material economically feasible.

Construction - If channel banks are constructed of compacted cohesive materials, compaction standards should be followed which would produce a soil structure least susceptible to swelling and shrinkage. Clay soils compacted dry of optimum have a relatively high swelling potential. Clay soils compacted wet of optimum have a relatively high shrinkage potential. Consequently, the most desirable compaction moisture content is very near the field optimum. Frequently the field optimum moisture content is slightly higher than the optimum moisture content determined by the California Field Impact laboratory compaction curves. For this reason, it is recommended that clay fills used for channel bank construction be compacted at a moisture content of 1 to 3 percent above the optimum as determined by the aforementioned laboratory test. Generally, clay soils should be compacted to a minimum relative compaction of 90 percent also based on the above referenced laboratory compaction test.

AREAS FOR FUTURE STUDY

The theory of tractive forces still contains many areas needing further study. The actual forces developed on channel boundaries should be more accurately defined. Lane's/[16] theory is based on a membrane analogy. The validity of this analogy is in question. Lane also defines the forces exerted on soil particles as a function of soil grain size. This relation does not appear adequate for fine-grained cohesive soils.

Developed tractive forces have been defined as a function of Manning's "n". Attempts should be made to define tractive forces as a function of more tangible factors. Einstein's bed-load functions might be one avenue for future study with respect to soil erosion. The effects of form roughness should also be considered as a possible cause of erosion.

The allowable tractive forces for fine-grained soils needs better definition and determination. Attempts in the past have been made to relate soil erosion to shear strength. A basic knowledge of soil mechanics indicates the validity of this assumption; however, all tests in the past have been made using soils of relatively low shear strength. Previous studies have also assumed an insitu constant shear strength when, in fact, the field soil shear strength varies as the water content varies. These areas merit additional research.

The foregoing items could be studied through the use of both laboratory and field experiments. A laboratory flume study is recommended to provide a better definition of developed tractive forces. An experimental channel constructed in the field could then be used to develop a relation between developed tractive forces and erosion or erosion rates.

Drying of clayey materials exposed on channel banks creates one of the most important problems associated with channel failures. Drying and rewetting of clay causes slaking and subsequent erosion. Drying also creates large tension cracks within a clay slope precipitating slope failures which account for the largest loss of materials and consequent property encroachment. It is recommended that plastics or other materials be studied to determine their suitability for sealing clay soils to eliminate the weathering effects. If clay soils could be kept at the natural moisture content, it is expected that slaking, cracking and loss of materials by slope disintegration could be reduced.

Other methods of increasing soil resistance to erosion should also be studied. Soil additives such as chemical stabilization, cement treating, or grouting are examples for possible research for practicability and economics.

Better methods of determining soil conditions along the alignment of a proposed channel improvement should be sought. As discussed with Mr. Ferguson of the Flood Control District, a statistical approach may prove practical. Conceivably, a statistical soil condition could be used for design. Channel-section design could then be altered to satisfy criteria of expected maintenance requirements. This is an area for which a sizable body of channel performance records correlated with soil types, hydraulic characteristics, flow volumes and velocities would be needed as a basis for statistical evaluation in channel design.

CONCLUDING REMARKS

It has been concluded that unlined channels are deteriorating in the Santa Clara Valley due to the creation of tractive forces greater than the ultimate tractive resistance of soils forming channel boundaries. Methods of estimating developed tractive forces and ultimate soil tractive force resistance have been proposed. The ultimate soil tractive force resistance of clay soils has been observed to be dependent on the soil water content. Cyclic wetting and drying of clays greatly reduces the tractive force resistance.

Slope instability has also been observed to be a major source of channel deterioration. Large chunks of channel banks have fallen into channel bottoms producing vertical banks. This type of slope failure is considerably different than the classical circular arc slope failure. Piping in fine-grained cohesionless soils near the toe of the channel bank has also decreased slope stability.

Determination of the soil conditions along a proposed channel alignment has been somewhat general in the past. Greater attention should be given to apparently minor details of the soil profile, particularly when soils are exposed to weathering elements and flowing water.

Based on this study and the conclusions mentioned above, recommendations have been given in this report for channel design to minimize soil erosion. In addition to the recommended tractive force method of analysis, recommendations have been given for the use of pervious gravel filter blankets to protect some soils against piping and for channel cross sectional shape. Recommendations have also been given for the compaction of clay soils. Many questions still remain unanswered regarding the design of stable channels. Some avenues of future research have been suggested in the latter section of this report.

APPENDIX B

EXISTING DESIGN TECHNIQUES

Several local governmental agencies were contacted to determine the existing design procedures used to limit or eliminate erosion of unlined channels. Following is a summary of different agencies' design techniques:

Santa Clara County Flood Control & Water District - In the past, the District has arbitrarily selected a maximum allowable channel design velocity of 8 ft/sec. More recently, a tractive force evaluation technique has been adopted. Developed tractive forces are mainly determined from formulas listed in Technical Release No. 25 issued by the Soil Conservation Service./[23] These formulas were developed by Emory Lane/[16] in 1955. Methods for determining allowable tractive forces for cohesionless soils are also taken from the manual. To evaluate the allowable tractive forces for cohesive soils, the District makes reference to an article prepared by Irving Dunn./[6] Figure 9 of Dunn's paper presents several design curves relating grain size and plasticity index to allowable tractive forces. The design curve recommended by Dunn is shown for sake of comparison in Figure 14 along with curves developed by the United States Bureau of Reclamation/[32] in 1953. Dunn's recommended values are considerably lower than those of the U.S.B.R. The District now uses the U.S.B.R. curve for no vegetation. As a check on the tractive force evaluation, the District makes reference to the allowable design velocities presented in Technical Release No. 25/[23] in Figure 6-1.

Alameda County Flood Control & Water Conservation District - The Alameda District constructs most channels with concrete-lined sections. Where unlined channels are developed, limiting velocities of 8 ft per second have been used. The County is presently experiencing problems with erosion and, particularly, slope stability. It is also current practice of the District to hire outside consultants before channel construction when erosion problems are anticipated.

Contra Costa County - The majority of channels constructed in Contra Costa County are lined. For unlined channels, the County uses the design approach presented in Technical Release No. 25 of the Soil Conservation Service./[23] Erosion problems have occurred, some of which have been defined as piping in fine-grained cohesionless materials. Slope stability problems are also common.

U.S. Dept. of Agriculture, Soil Conservation Service - The Soil Conservation Service has developed, for their design purposes, Technical Release No. 25./[23] Technical Release No. 25 contains allowable design velocities developed by both the USSR/[27] and by Fortier and Scobey./[9] These allowable design velocities are related to soil grain size and have been proven over long periods of successful channel design. The major emphasis has been placed on non-cohesive soils. Only limited allowable-channel-velocity criteria for cohesive soils are available.

Technical Release No. 25 also presents the latest tractive force design analyses organized by Lane/[16] in 1955. Further attempts to define allowable tractive forces for cohesive materials have recently been made. The most recent Technical Release No. 25 design manual, edition dated 9 December 1968, attempts to relate unconfined compressive strength to allowable tractive power. (Tractive power is defined as the product of the allowable tractive force and velocity.) Based on discussions with Dr. George Kalkanis and others from the Soil Conservation Service, it is our understanding that some problems are still developing in cohesive channels despite attempts to relate erodibility to unconfined compressive strength. Problems are also occurring with slope stability.

United States Army Corps of Engineers - The Army Corps of Engineers in San Francisco has established allowable velocity criteria for design. Allowable velocities of 6 ft per second are used for coarse sands and gravel, and of 5 ft per second for fine sands and silts. If higher velocities are needed, the channels are always lined. Clay channels are, for the most part, also lined.

TABLE I

SOIL CLASSIFICATIONS & LABORATORY TESTS

LOCATION & SAMPLE NO.		LOCATION	DEPTH ** (ft.)	CLASSIFICATION	LIQUID LIMIT	PLASTIC INDEX	WATER CONTENT (%)	DRY DEN-SITY PCF	UNCON-FINED COMPRES-SIVE STRENGTH	DEGREE OF EROSION
Ross Ck.	1	1000' west of Jarvis	4	Lt. brn. sandy clayey silt (ML)	21	5				Moderate
	2	1000' west of Jarvis	10	Lt. brn. sandy gravel (GW)						Slight
	3	200' west of Cherry	9	Lt. brn. sandy clay (CL)						Moderate
	4	200' west of Cherry	10	Lt. brn. sandy clay (CL)	29	11				Moderate
Sierra Creek	1-1	Sta 1 + 00	10	Lt. brn. sandy clay (CL)	44	26	9	106	14,290	Moderate
	1-2	Sta 1 + 00	8	Dk. brn. sandy clay (CL)	38	20	19	102	2,170	Slight
	2	Sta 10 + 00	8	Lt. brn sandy clayey silt (ML)	36	12				Slight to Moderate
	3	Sta 12 + 00	8	Sandy gravel (GW)						Slight
	4-1	Sta 44 + 00	9	Brn. clayey sand (SC)	32	14	12	97	740	Slight to Moderate
	4-2	Sta 44 + 00	9	Brn. clayey sand (SC)						Slight to Moderate
Smith Creek	3-1	100' north of Elam	12	Lt. brn. clayey gravel (GC)						Very Slight
	3-2	100' north of Elam	8	Lt. Brn. clayey sand (SC)						Very Slight
	4-1	500' north of Elam	9	Brn sandy clay (CL)	37	18	11			Slight to Moderate
Golf Creek	1	Between Redwood & Almaden *		Dense sandy gravel (GW)						Moderate
	2	Between Redwood & Almaden *		Brn Sandy Clay (CL)						Moderate
Calabazas	1-1	300' east of Kifer	8	Lt. gray-Brn., V. clayey sand (SC)	32	15	20	105	2,510	Severe
	1-2	300' east of Kifer	2	White silty clay (CL)	42	24	19	104	10,940	Severe
	1-3	300' east of Kifer	5	Tan clayey silty sand (SM)						Severe
	2-1	600' west of Kifer	5	White sandy clay (CL)	32	18				Severe
	2-2	600' west of Kifer	8	Brn-gray silty clay (CL)	37	21	18	106	5,030	Severe
	3-1	500' north of Monroe	5	Brn. V. clayey sand (SC)	39	23				Severe
	3-2	500' north of Monroe	8	Lt. Brn.-gray sandy clay (CL)	43	26	19	103	9,070	Severe
	4-1	300' north of Monroe	5	White sandy clay (CL)	37	20				Severe
	4-2	300' north of Monroe	8	Lt. Brn.-gray clayey sandy silt (ML)	39	20	25	96	2,610	Severe
	5-1	1000' north of Benton	8	Lt. Brn.sandy clay(CL)	34	16	19	98	2,170	Moderate
	6-1	1000' west of Pomeroy	8	Lt. Brn. clayey sand (SC)						Severe
	6-2	1000' west of Pomeroy	14	Dk. Brn. gravelly clay (CH)	56	35				Severe
	7-1	400' west of Pomeroy	8	Lt. Brn. clayey sand (SC)			9	130	5,960	Slight Moderate
	7-2	400' west of Pomeroy	12	Dk. Brn. gravelly clay (CL-CH)	48	29	14	107	12,120	Slight Moderate
Sunnyvale East Channel		El Camino Real to Bayshore Freeway*	0-3	Blk silty clay						Severe
			3-9	Lt. Gray-Brn. silty Clay with lime cementation						Severe
			9	Lt. Brn. clayey sand						Severe

* Idealized soil conditions

** Depth Below Surface

TABLE II

VELOCITIES AND TRACTIVE FORCES OF SANTA CLARA COUNTY FLOOD CONTROL AND WATER DISTRICT CHANNELS

CREEK AND SAMPLE	MAXIMUM FLOW (CFS) (Q)	DEPTH AT MAXIMUM FLOW (FT) d	CHANNEL SECTION AS BUILT b$^{(1)}$	z$^{(2)}$	s$^{(3)}$	VELOCITIES$^{(4)}$ (ft/sec) Max.developed	Allowable	DEVELOPED TRACTIVE FORCE (PSF) BOTTOM (T_b)	SIDE (T_s)	ALLOWABLE TRACTIVE FORCE (PSF) BOTTOM (T_b)	SIDE (T_s)
Ross	760	5.1	8'	1-1/2:1	.0039	9.5					
1							4.0	0.770	0.645	0.40	0.40
2							3.1	0.740	0.619	0.25	0.10
3							4.0	0.770	0.645	0.44	0.44
4							4.0	0.770	0.645	0.36	0.36
Sierra		3.0 (assumed)	6'	1-1/2:1	.0059	8.0					
1-1							4.0(max)	0.710	0.582	0.38	0.38
1-2							4.0	0.710	0.582	0.50	0.50
2							4.0	0.710	0.582	0.36	0.36
3							2.7	0.781	0.641	0.32	0.13
4-1							4.0	0.710	0.582	0.50	0.50
4-2							4.0	0.710	0.582	0.12	0.12
Smith	300 (est)	4.0	8'	1-1/2:1	.0013	4.0					
3-1							4.1	0.206	0.168	0.56	0.22
3-2							4.0	0.216	0.176	0.005	0.005
4-1							4.0	0.216	0.176	0.45	0.45
Golf	250	2.5	8'	1-1/2:1	.005	7.7					
gravels							4.0-5.0	0.740	0.585	0.40	0.16
clays							2.5-4.0	0.535	0.422	0.20	0.20
Calabazas	800	4.8	10'	1-1/2:1	.004	9.7		0.746	0.612		
1-1							4.0	0.768	0.560	0.17	0.17
1-2							5.5			0.50	0.50
1-3							4.0			0.17	0.17
2-1							4.0			0.49	0.49
2-2							5.5			0.49	0.49
3-1							5.5			0.28	0.28
3-2							5.5			0.42	0.42
4-1							5.5			0.42	0.42
4-2							5.5			0.44	0.44
5-1							4.0			0.42	0.42
6-1							4.0			0.20	0.20
6-2							5.5			0.50	0.50
7-1							4.0			0.20	0.20
7-2							5.5			0.50	0.50

b$^{(1)}$ - bottom width (feet)

z$^{(2)}$ - side slope (horz/vert)

s$^{(3)}$ - bottom slope (feet/foot)

(4) - permissible velocities obtained from charts and table developed by Forteer and Scobey, shown as Fig. 6-1 in TR25 of the SCS for suspended load

SEDIMENTATION
IN
SOUTH SAN FRANCISCO BAY BELOW
DUNBARTON BRIDGE

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March 8, 1972

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SEDIMENTATION
IN
SOUTH SAN FRANCISCO BAY BELOW
DUNBARTON BRIDGE

Introduction

An estuary is the terminous of land drainage. It is the transition region from unidirectional varying stream flows to the ocean, and it is subjected to the effects of ocean tides, salinity, and temperature, to the on-shore and off-shore winds associated with lands bordering oceans and to cyclonic winds associated with storms, and to the discharge of wastes, as well as to the amounts of stream discharges and their constituents. The ways these factors affect an estuary depend on its bed and shoreline configuration and on the configuration of the land around its shores.

These configurations change continually. Geologic processes, the most important of which is land erosion, change estuary configurations even without the presence of man. Human endeavors, however, have greatly accelerated changes in estuary configurations by altering the rates of land erosion, by reducing tideland areas, and by deepening channel and harbor areas. At the same times major man-made changes in fresh water inflows and waste discharges burden estuary waters and their biota so that deterioration of an estuary by man is a real possibility. Clearly, deliberate modifications of an estuary's configuration and of its fresh water and erosion product inflows, and waste discharges to an estuary must be planned with their short and long range effects considered to the extent current knowledge allows if man is to achieve the most desirable environment his means afford. The evident effects of sediment on Bay waters resulting from its transport and accumulation are worth elaborating.

The accumulation of sediment can so alter man-modified bed and shoreline configurations as to negate their benefits in a short time. The Martinez Yacht Harbor, for example, fills with sediment at rates of several feet a year. A three mile length of Mare Island Strait, widened to 1000 feet to accommodate Navy needs, shoals up to two million cubic yards each year, and requires approximately one-fourth of the total maintenance dredging conducted in the San Francisco Bay-Delta system. Alterations of the Bay or marshes that cause accumulation of sediments require continuing expensive maintenance. Shoaling is therefore a major factor in selecting a drainage system design.

Suspended sediments limit the penetration of sunlight into estuarial waters with three significant results. It appears that with existing municipal and industrial waste discharges to the system, including discharges from agriculture, there are abundant nutrients for growths of photosynthetic aquatic organisms. The limited intensities of light resulting from the presence of suspended sediment, however, limits the rate of multiplication of suspended algae, relative to the rate of their loss due to consumption by higher organisms and by transport to the sea, so that nuisance level concentrations of algae occur only occasionally now. If the concentration of suspended sediment is reduced in the water without at the same time making some nutrient such as nitrogen limiting, then algae can be expected to cause unsightly and odorous floating mats and to deplete oxygen to fish-killing levels. Rooted plants will also grow farther from shore.

Suspended sediment has a very large sorptive capacity, and many dissolved toxic materials are adsorbed from solution and are strongly held on mineral particles. Adsorbed materials are carried with the particles to deposits. This sorptive capacity has provided a significant part of the assimilative capacity of the system for toxic wastes and has reduced the effects of such materials on aquatic life. These last two effects are beneficial. Before deliberately reducing the concentration of suspended sediment in Bay waters it will be necessary to limit some nutrient and to restrict the discharge of toxic materials if desirable water quality for aquatic life is to be maintained.

The third consequence of the limited penetration of light, that is the absorption of most of the sunlight near the water surface, is enhanced warming of the surface water. Higher surface temperatures reduce the rate of oxygen transfer from air to water and tend to stabilize the water so that mixing of surface and deeper water is impeded; both effects reduce oxygen transfer from the atmosphere to the bed. This warming probably is significant only when wave action and vertical circulation are limited.

It can result from any suspended or dissolved material that reduces water transparency.

Muddiness is claimed to reduce the aesthetic qualities of estuary waters. Frequent muddiness is a natural condition in the shallow portions of the system and in the region where fresh and salt waters mix, however, and does not seem to impair the uses of these portions of the estuary for boating, fishing, or aesthetic enjoyment. Contact sports may be limited where muddiness is pronounced.

An understanding of the factors affecting sediments in the bay and sloughs south of the Dunbarton bridges is important to planning development of the region because of the effects of sediment described above. The sources of sediments, their character, sediment transportation processes, and the overall transport pattern are described within the limits of present knowledge in the following sections to aid in planning development of the floodlands.

Character of Sediments

Deposited sediment in South Bay and the surrounding sloughs is generally described as "Bay mud." It contains 20 to 60 percent clay minerals and 8 to 10 percent or more volatile solids (organic matter, carbonates, and water of crystalization) which make older deposits gel and give the material its "mud" character. The remainder of the material is silt and fine sand. Patches of shells occur, and fragments of broken shells are common constituents of sediment deposits. Material in the marshlands contain plant roots, and the organic matter content in such areas is variable.

The best quantitative descriptions of sediment character in South Bay are those provided in reports on the Comprehensive Study of San Francisco Bay conducted by members of the Sanitary Engineering Research Laboratory, College of Engineering, and School of Public Health, University of California, Berkeley, in fifteen volumes. The following data are taken from the reports on sampling and sample analyses for South San Francisco Bay.

The locations of sampling stations are shown in Figure 1 and Table I (1). The stations numbered 5 through 9 are appropriate to this study; unfortunately they are all located in or next to navigation channels and do not provide information on sediment in the shallow mud flats. The information from these data is useful, and inferences of the character of shallow areas can be made from other information.

Physical and chemical characteristics of the sediment samples are presented in Tables II through VII (2). The last eight columns of these tables show that the deposits in the area contain small amounts of well stabilized organic matter. Both the hexane extractible material (HEM) and the organic carbon contents indicate the amounts of organic material. The presence of sulfides in most of the samples indicates that the sediment is generally anaerobic, and the carbon to nitrogen ratios indicate the well stabilized nature of the material. The BOD values in the last column indicate the maximum five day oxygen demand that could be expected if these materials were dispersed in Bay waters.

The color of the sediment is determined largely by the oxidation state of iron. After aeration the material has a light brown color. As micro-organisms deplete oxygen, iron becomes reduced to a greenish-gray color, and with continuing reduction the black ferrous sulfide formed gives deposits a dark color. The dark color is more pronounced with increased organic matter and longer length of time after the material is deposited. The grey and black descriptions in Tables II and III together with the presence of sulfides indicate the anaerobic condition of these samples.

Particle size distributions are shown in the tables as fractions by weight of sand (greater than 50 microns diameter), silt (2 to 50 microns) and clay (less than 2 microns diameter). These data are presented graphically in the triaxial plot in Figure 2. The grouping of data in Figure 2 shows that the composition of sediment near the channels changes with time. The clay and silt fractions were greater in May than those of samples taken the previous October, January, and March. An explanation for this change is presented in the next section.

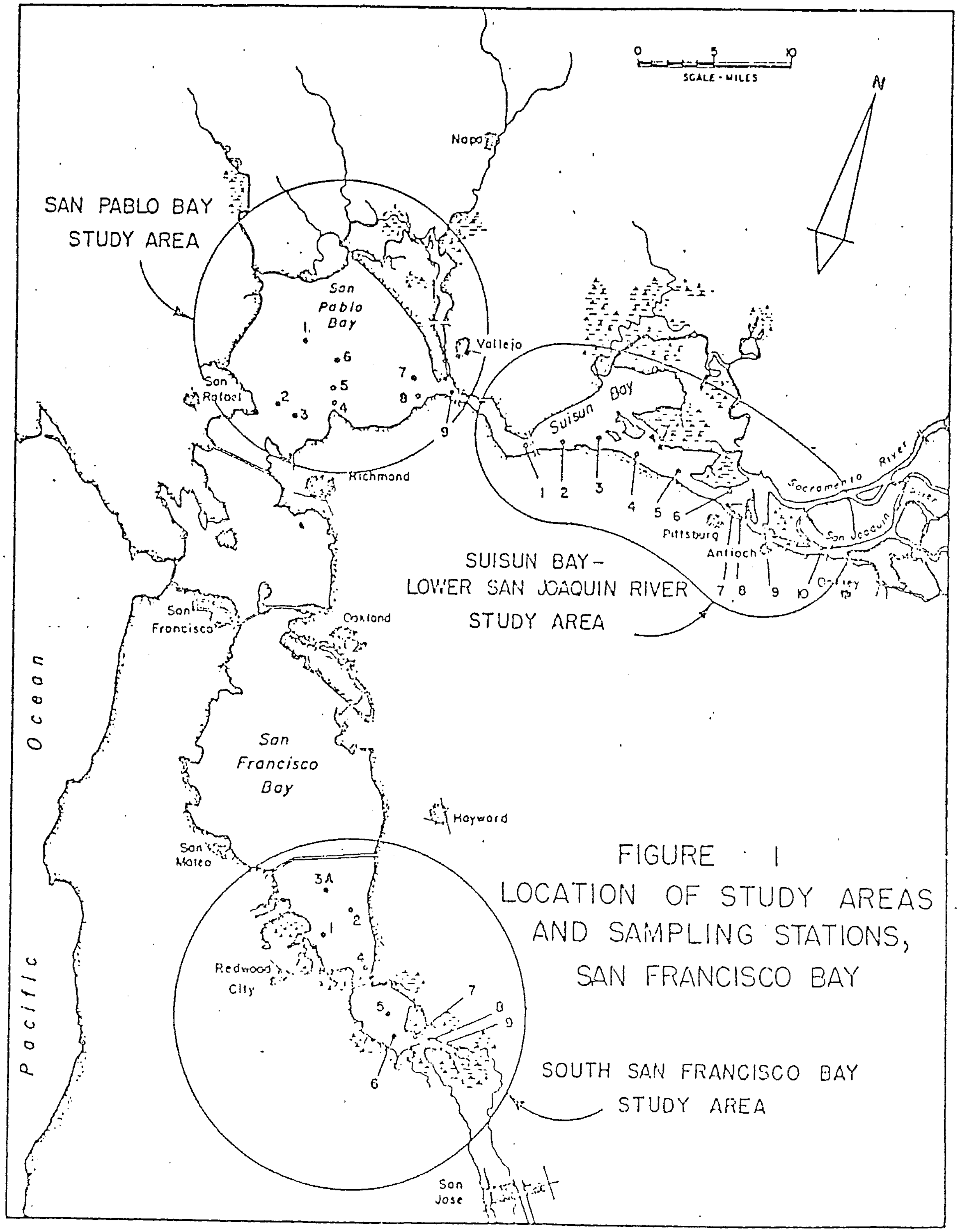


FIGURE I
LOCATION OF STUDY AREAS
AND SAMPLING STATIONS,
SAN FRANCISCO BAY

From "A Comprehensive Study of San Francisco Bay 1961-62"

TABLE 1

SAMPLING STATIONS
SOUTH SAN FRANCISCO BAY

Station Number	Description*	Location*	
		N Lat	W Long
1	East 0.8 mile from light Fl. G 2-1/2 sec. on Redwood Creek	37°32'27"	122°10'33"
2	100 yds. astern of the wreck off Coyote Hills Slough	37°33'09"	122°09'28"
3A	North 55° east 0.8 mile from buoy R "10" at Steinbergen Slough	37°34'03"	122°11'48"
4	Main channel 200 yards northwest of northeast lift tower of Dumbarton Highway Bridge	37°30'26"	122°07'07"
5	Midchannel at buoy "3" near Calaveras Point	37°28'42"	122°04'32"
6	Midchannel at buoy "5" off Calaveras Point	37°27'32"	122°03'00"
7	Midchannel in Coyote Creek 100 feet west of P.G.E. crossing near mouth of Alviso Slough	37°27'49"	122°01'28"
8	200 feet west of piling at San Jose secondary outfall	37°27'54"	121°59'40"
9	Midchannel 200 feet west of SFRR bridge over Coyote Creek	37°27'38"	121°58'28"

*Ref: U.S.C. and G.S. Chart No. 5531 corrected to 6/17/61.

From "A Comprehensive Study of San Francisco Bay 1961-62"

TABLE II

PHYSICAL AND CHEMICAL SEDIMENT CHARACTERISTICS, SOUTH SAN FRANCISCO BAY

11 July 1961

Station	Sample	Water Depth	Temp.	Description	Moisture	Volatile Matter	H E M	Total Sulfide	Total Nitrogen	Carbon		Ratio Org. C: Total N	BOD
										Total	Organic		
		ft	°C		%	%	mg/kg	mg/g	mg/g	%	%		mg/g
1	A	45	21.8	Silty Clay		11.0		0.46	1.48	1.46	0.83	5.6	0.79
	B	45	21.7	Silty Clay		10.3		0.92	1.34	1.18	1.09	8.1	2.4
2	A	10	22.6	Shell and Silt		18.5		0.63	0.99	4.09	0.53	5.4	1.0
	B	10	23.0	Shell and Silt		21.0		0.28	1.20	4.94	0.41	3.4	0.80
3A	A	19	21.1	Dark Grey Silt		9.9		0.43	1.33	1.40	1.66	12	2.4
	B	19	20.7	Dark Grey Silt		9.2		0.57	1.28	1.15	1.06	8.3	1.9
4	A	45	23.3	Sandy Silt with Shells		8.0		0.28	1.12	1.10	0.93	8.3	1.7
	B	45	23.0	Sandy Silt with Shells		7.8		0.51	1.04	1.10	0.82	7.9	1.6
5	A	45	23.2	Sandy Silt with Shells		24.7		0.96	1.16	5.48	0.82	7.1	2.4
	B	45	22.7	Sandy Silt with Shells		10.8		0.55	1.39	1.40	1.20	8.6	2.7
6	A	16	23.0	Grey Silty Clay		11.0		0.10	1.67	1.55	1.39	8.3	1.7
	B	16	23.5	Grey Silty Clay		10.6		0.16	1.53	1.54	1.39	9.1	1.8
7	A	17	25.0	Silty Clay		11.5		0.70	1.65	1.47	1.35	8.2	4.8
	B	17	24.2	Silty Clay		11.4		0.73	1.60	1.41	1.36	8.5	2.7
8	A	5	23.5	Sludge		11.5		3.59	1.92	1.46	1.41	7.3	7.1
	B	5	23.5	Sludge		11.7		4.00	2.27	1.81	1.72	7.6	10
9	A	16	25.2	Black Sludge, H_2S Odor		11.4		3.92	1.91	2.02	1.99	10	0.77
	B	16	24.4	Black Sludge, H_2S Odor		12.0		4.67	1.87	1.68	1.59	8.5	7.8

From "Appendixes to A Comprehensive Study of San Francisco Bay 1961-62"

TABLE III

PHYSICAL AND CHEMICAL SEDIMENT CHARACTERISTICS, SOUTH SAN FRANCISCO BAY

24 August 1961

Station	Sample	Water Depth	Temp.	Description	Moisture	Volatile Matter	H E M	Total Sulfide	Total Nitrogen	Carbon		Ratio Org. C: Total N	BOD
		ft	°C		%	%	mg/kg	mg/g	mg/g	Total %	Organic %		mg/g
1	A	40	21.2	Grey Silty Clay with Thin Strata of Sand at Surface	59.0	8.8		0.83	1.56	1.15	1.02	6.5	2.6
	B	40	21.0	Grey Silty Clay with Thin Strata of Sand at Surface	57.9	8.7		1.90	1.52	1.12	1.04	6.8	2.7
2	A	10	20.5	Grey Sandy Silt with Shells	45.3	11.0		0.27	1.07	5.69	1.52	14	0.85
	B	10	20.8	Grey Sandy Silt with Shells	50.3	13.3		0.37	1.11	4.33	0.55	5.0	0.68
3A	A	16	20.8	Grey Silt with Traces of Sand	53.1	9.0		0.51	1.35	1.61	1.12	8.3	2.4
	B		20.8	Grey Silt with Traces of Sand	50.7	7.7		0.55	1.31	1.24	1.01	7.7	2.3
4	A	40	20.9	Grey Sandy Silt	34.0	6.5		0.71	0.82	1.16	0.61	7.4	0.85
	B	40	20.9	Grey Sandy Silt	35.0	9.4		0.31	0.81	1.79	0.03	0.37	0.80
5	A	30	21.4	Grey Sandy Silt	54.4	3.0		0.66	1.72	1.30	1.14	6.6	2.5
	B	30	21.1	Grey Sandy Silt	55.7	8.5		0.62	1.58	1.34	1.20	7.6	3.1
6	A	15	21.5	Grey Sandy Silt	56.9	8.9		0.65	2.09	1.38	1.28	6.1	2.2
	B	15	21.4	Grey Sandy Silt	54.3	9.2		0.51	1.82	1.65	1.32	7.3	1.7
7	A	15	21.5	Dark Grey Silt	56.6	9.2		0.88	1.97	1.60	1.41	7.2	3.0
	B	15	21.6	Dark Grey Silt	56.3	8.8		0.94	2.06	1.60	1.51	7.3	3.3
8	A	12		Black Mud	65.5	10.6		6.32	2.94	1.88	1.71	5.8	13
	B	8		Black Mud	59.9	10.1		3.84	2.63	1.87	1.70	6.5	8.7
9	A	15		Black Mud	66.7	9.4		3.06	2.26	1.58	1.52	6.7	11
	B	15		Black Mud	66.4	9.5		4.94	2.22	1.55	1.51	6.8	7.7

From "Appendixes to A Comprehensive Study of San Francisco Bay 1961-62"

TABLE IV

PHYSICAL AND CHEMICAL SEDIMENT CHARACTERISTICS, SOUTH SAN FRANCISCO BAY

24 October 1961

Station	Sample	Water Depth	Temp.	Particle Size Distribution % by Weight			Moisture	Volatile Matter	H E M	Total Sulfide	Total Nitrogen	Carbon		Ratio Org. C: Total N	BOD
		ft	°C	Sand	Silt	Clay	%	%	mg/kg	mg/g	mg/g	Total %	Organic %		mg/g
1	A	40	17.3	48	22	30	56.8	8.8		0.46	1.35	1.26	1.08	8.0	3.4
	B		17.5				56.9	8.9		0.32	1.37	1.32	1.13	8.3	2.9
2	A	11	17.0	45	27	28	50.8	7.3		0.65	1.15	0.94	0.88	7.7	1.4
	B	11	17.0				51.7	7.5		0.54	1.19	1.10	1.04	8.7	1.7
3A	A	12	17.1	46	21	33	54.3	8.2		0.20	1.26	1.10	1.01	8.0	2.2
	B	12	17.2				58.6	8.4		0.54	1.36	1.12	1.03	7.6	3.9
4	A	45	17.2	45	22	33	44.8	6.5		0.13	1.08	1.01	0.88	8.2	1.2
	B		17.2				34.0	6.3		0.07	0.80	1.26	0.50	6.3	1.1
5	A	40	17.5	44	21	35	56.2	12.0		0.85	1.72	2.59	0.90	5.2	2.8
	B		17.0				54.9	8.6		0.22	1.53	1.38	1.20	7.8	2.9
6	A	18	17.2	44	19	37	56.7	8.9		0.71	1.83	1.40	1.32	7.2	2.0
	B		17.0				57.1	9.2		0.84	1.74	1.44	1.33	7.6	2.6
7	A	16	17.0	45	15	40	51.8	9.8		1.48	1.63	1.89	1.32	8.1	3.6
	B	16	16.5				45.6	8.1		1.86	1.54	1.69	1.58	10	3.4
8	A	12	17.8	69	6	25	61.4	9.3		3.32	2.15	1.57	1.47	6.8	9.7
	B	12	17.6				58.8	8.9		3.56	1.93	1.50	1.39	7.2	8.6
9	A	9	18.2	64	13	23	55.8	9.0		2.37	2.50	1.93	1.79	7.2	13
	B	9	18.0				48.9	7.7		2.59	1.79	1.69	1.55	8.7	8.6

From "Appendixes to A Comprehensive Study of San Francisco Bay 1961-62"

TABLE V

PHYSICAL AND CHEMICAL SEDIMENT CHARACTERISTICS, SOUTH SAN FRANCISCO BAY

8 January 1962

Station	Sample	Water Depth	Temp.	Particle Size Distribution % by Weight			Moisture	Volatile Matter	H E M	Total Sulfide	Total Nitrogen	Carbon		Ratio Org. C: Total N	BOD
												Total	Organic		
		ft	°C	Sand	Silt	Clay	%	%	mg/kg	mg/g	mg/g	%	%		mg/g
1	A	42	10.1	47	17	36	58.2	7.7	475	0.90	1.38	1.32	1.28	9.3	3.0
	B	42	10.5				60.9	8.0	643	0.85	1.34	1.23	1.15	8.6	2.8
2	A	9	10.1				50.4	7.0	644	0.30	1.10	3.05	0.38	3.5	1.3
	B	9	10.1				43.6	6.3	234	0	1.08	3.95	1.07	9.9	1.2
3A	A	21	10.1	53	14	33	57.7	7.5	694	0.20	1.39	1.27	1.06	7.6	2.1
	B	21	10.2				56.8	7.4	684	0.69	1.38	1.42	1.26	9.1	2.2
4	A	48	10.3	80	9	11	43.6	5.9	307	0.35	0.81	0.94	0.76	9.4	1.4
	B	48	10.2				42.3	5.4	455	0.53	1.00	1.60	0.06	0.60	1.5
5	A	47	10.8	43	17	40	58.7	8.4	494	0.31	1.58	1.24	1.10	7.0	3.3
	B	47	10.5				57.1	7.9	566	0.64	1.51	1.32	1.17	7.8	2.8
6	A	16	10.8	44	17	39	57.1	8.6	267	0.59	1.92	1.52	1.52	7.9	2.8
	B	16	10.0				56.6	8.6	342	0.64	1.75	1.38	1.29	7.4	2.5
7	A	14	10.1	56	9	35	55.2	8.4	63	0.98	1.70	1.46	1.40	8.2	2.9
	B	14	10.3				54.5	8.3	271	1.07	1.68	1.40	1.34	8.0	3.2
8	A	17	10.9	37	18	45	63.0	9.0	430	2.87	2.05	1.40	1.33	6.5	6.8
	B	17	11.0				64.9	10.0	526	3.48	2.23	1.50	1.43	6.4	7.0
9	A	12	11.8	43	13	44	62.4	9.3	475	2.79	2.17	1.70	1.61	7.4	9.6
	B	12	11.2				59.6	8.6	279	2.26	1.97	1.49	1.44	7.3	6.7

From "Appendixes to A Comprehensive Study of San Francisco Bay 1961-62"

TABLE VI

PHYSICAL AND CHEMICAL SEDIMENT CHARACTERISTICS, SOUTH SAN FRANCISCO BAY

1 March 1962

Station	Sample	Water Depth	Temp.	Particle Size Distribution % by Weight			Moisture	Volatile Matter	H E M	Total Sulfide	Total Nitrogen	Carbon		Ratio Org. C: Total N	BOD
												Total	Organic		
		ft	°C	Sand	Silt	Clay	%	%	mg/kg	mg/g	mg/g	%	%		mg/g
1	A	35	10.0	66	10	24	61.5	9.2	104	0.38	0.08	1.13	1.13	140	0.63
	B	35	10.0				63.1	9.5	462	0.37	1.42	1.38	1.38	9.7	0.76
2	A	5	9.8				86.4	12.2		0	0.59	3.16	0.66	11	
	B	5	9.8				84.0	11.9		0	0.45	3.76	0.92	21	
3A	A	18	10.5	50	15	35	66.7	9.4	429	0.54	1.19	1.29	1.26	11	0.47
	B	18	10.5				66.3	9.2	782	0.43	1.41	1.03	0.99	7.0	0.50
4	A	44	10.0	52	15	33	50.4	7.4		0.18	0.04	1.02	0.71	180	
	B	44	9.8				40.4	5.6		0.07	1.02	1.33	0.19	1.8	
5	A	20	10.8	50	10	40	57.0	9.1	825	0.20	1.48	1.29	1.18	8.0	0.24
	B	20	10.8				59.3	9.2	35	0.21	2.82	1.39	1.27	4.5	0.19
6	A	15	9.7	48	12	40	63.0	9.7	429	0.05	2.02	1.59	1.42	7.0	0.16
	B	15	9.7				57.8	9.2	269	0.28	0.12	1.32	1.24	1.0	0.24
7	A	10	9.8	52	11	37	58.7	9.5	389	0.98	0.17	1.44	1.34	79	0.26
	B	10	9.8				56.3	9.1	556	0.77	2.12	1.34	1.24	5.9	0.23
8	A	18	10.2	41	12	47	70.1	10.5	328	4.3	1.38	1.62	1.56	11	0.93
	B	18	10.0				65.8	10.2	301	4.3	2.18	1.60	1.53	7.0	0.88
9	A	9	11.4	38	11	51	69.2	10.2	1130	5.7	2.24	1.65	1.61	7.2	0.87
	B	9	11.0				69.9	10.5	1260	5.8	2.32	1.80	1.73	7.5	1.2

From "Appendixes to A Comprehensive Study of San Francisco Bay 1961-62"

TABLE VII

PHYSICAL AND CHEMICAL SEDIMENT CHARACTERISTICS, SOUTH SAN FRANCISCO BAY

1 May 1962

Station	Sample	Water Depth	Temp.	Particle Size Distribution % by Weight			Moisture	Volatile Matter	H E M	Total Sulfide	Total Nitrogen	Carbon		Ratio Org. C: Total N	BOD
												Total	Organic		
		ft	°C	Sand	Silt	Clay	%	%	mg/kg	mg/g	mg/g	%	%		mg/g
1	A	42	16.0	19	31	50	62.2		966	0.09	1.47	1.21	1.14	7.8	2.9
	B	42	15.9				60.2		914	0.55	1.16	1.18	1.08	9.3	3.7
2	A	8	16.2	27	37	36	51.8		510	< 0.01	1.19	1.10	0.89	7.5	1.4
	B	8	16.2				51.0		329	0.03	1.05	1.42	0.84	8.0	1.2
3A	A	16	15.0	15	33	52	59.0		558	0.08	1.55	1.27	1.17	7.6	1.8
	B	16	15.2				59.3		754	0.06	1.50	1.24	1.11	7.4	1.8
4	A	45	16.0	36	31	33	43.4		367	0.04	1.02	1.75	0.66	6.5	1.2
	B	45	15.9				49.0		203	0.05	1.13	1.72	1.32	12	1.5
5	A	42	15.8	20	22	58	61.4		726	0.36	1.73	1.28	1.14	6.6	2.9
	B	42	15.8				60.2		147	0.05	1.59	1.35	1.21	7.6	2.6
6	A	14	16.0	22	21	57	62.1		299	0.03	1.98	1.60	1.35	6.8	2.2
	B	14	16.0				62.2		473	0	2.07	1.64	1.42	6.9	3.4
7	A	14	16.3	16	29	55	59.1		422	1.8	1.73	1.68	1.49	8.6	4.1
	B	14	16.3				56.6		902	0.24	1.82	1.59	1.39	7.6	4.5
8	A	13	17.1	11	21	68	68.1		1600	2.8	2.15	1.52	1.42	6.6	8.9
	B	13	17.0				64.8		554	1.6	1.95	1.40	1.33	6.8	6.6
9	A	7	17.0	16	25	59	69.2		1140	5.2	2.31	1.65	1.57	6.8	10
	B	7	17.1				69.5		638	4.8	2.57	1.72	1.62	6.3	12

From "Appendixes to A Comprehensive Study of San Francisco Bay 1961-62"

NOTE: NUMBERS ON GRAPH REFER TO SAMPLING STATIONS. SEE TEXT DISCUSSION.

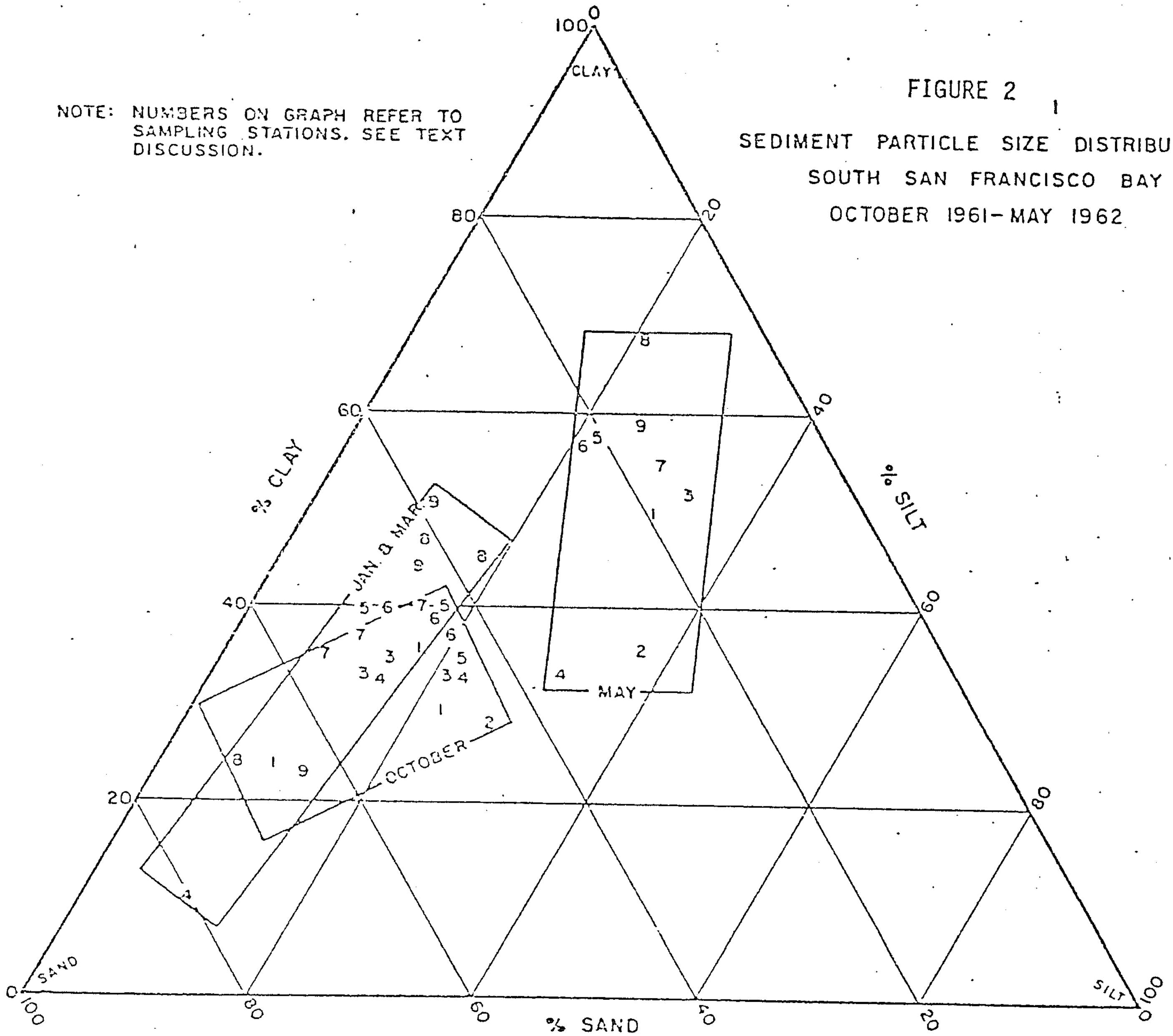


FIGURE 2

SEDIMENT PARTICLE SIZE DISTRIBUTION
SOUTH SAN FRANCISCO BAY
OCTOBER 1961 – MAY 1962

From "A Comprehensive Study of San Francisco Bay 1961-62"

Cation exchange capacities were not determined for the South Bay samples. Those in the lower Bay (stations 1 to 6) had values ranging from 8 to 66 milli-equivalents per hundred grams with median values around 25 me/100g. Similar values should obtain from South Bay sediments. The cation exchange capacity varies with the clay content, and is indicative of the sorptive capacity of the sediments. The clay minerals that comprise Bay mud are principally illite, montmorillonite, and kaolinite.

The important properties of these sediment materials to development of the area and to water quality are the very small sizes of the individual mineral particles and their large reactive surface areas per unit weight. These properties determine the mode of transport of these materials and the character of an accumulated deposit.

Sediment Transportation Processes

The fine mineral particles are from soils formed on the upland slopes. Some may have paused in secondary deposits on valley floors during intermittent transportation to the Bay by overland flows. The finer particles are carried in suspension as wash load during periods of stream flow with the largest amounts carried during the high runoffs that follow winter storms. The very small sizes of the individual particles give them low settling velocities, and turbulence of stream flows is sufficient to keep them suspended. The clay particles move as individuals because a cloud of cations, attracted by the negative surface charge typical of these minerals, forms around each particle. When one particle approaches another so that these clouds intermingle, the mutual repulsion of the similarly charged ion clouds results in mutual repulsion of the particles. The particles remain dispersed.

When stream waters mix with ocean waters in the Bay to the extent of one part or more ocean water to 33 parts fresh water, however, the concentration of cations everywhere in the suspension is close enough to the concentration near the clay particle surfaces that particles can come in close proximity to each other before feeling the increased repulsion due to the ion clouds. A universal very short range attractive force, called the London-Van der Waals force, then dominates the mutual repulsion and suspended particles are mutually cohesive. The cohesion is not strong, but it is strong enough to hold mineral particles together under many hydraulic conditions in the Bay and sloughs.

Repeated collisions of suspended cohesive particles result in the formation of large aggregates of minerals compared to the sizes of the individual mineral particles. Both cohesion and repeated collisions are requisite to the formation of aggregates. The aggregates can be rendered by high velocity gradients and reformed by the collisions facilitated at lower velocity gradients, repeatedly. The size, density, shear strength, and rate of growth of these aggregates determine their transportation under the hydraulic conditions of the system.

A description of these aggregates has been found by rheologic studies of sediment suspensions that is useful in describing estuarial sediment transportation processes (3). Consider first an assemblage of mineral particles formed by the addition of one particle at a time to the growing aggregate. Such an aggregate would have a structure like that of a popcorn ball, with pore space between the individual more or less randomly oriented particles filled with water, and with a density, including the pore water in the volume of the aggregate, much lower than that of an individual particle. The aggregate would have a larger size than that of the individual particles. This aggregation is named a "primary particle aggregate."

Suppose that a suspension of primary particle aggregates is subjected to internal shearing so that these aggregates collide, and the particles in the area of contact of the aggregates cohere. A succession of collisions would lead to an aggregation of primary particle aggregates that would include additional pore volume in the large pore spaces between primary aggregates. This aggregation of particle aggregates is named a "first order aggregate." Suspension of such aggregates can form "second order aggregates" from collisions and cohesion of first order aggregates, and so on. Each order of aggregation has lower density, lower shear strength and larger dimensions than its immediate precursor. Densities and shear strengths of aggregates having up to six orders of aggregation of San Francisco Bay sediments have been measured (3).

The settling velocity of an aggregate is proportional to its diameter squared, so that a large aggregate, relative to the size of a mineral particle, will settle much faster even though its density is less. Higher order aggregates can form quickly in waters subjected to gentle shearing, such as that passing through the piling under piers, and high rates of deposition are observed in areas in the lee of piers. The area west of the Benicia Arsenal pier, for example, filled 29 feet in the ten years following the extension of the pier.

When aggregates settle to the bed and accumulate, a layer having one order higher than that of the depositing aggregate is formed. If accumulation continues the deposit will become thick enough to crush the lower aggregates, so that below a centimeter or so a bed is more dense. Further, a slow gelling occurs in the deposit that becomes pronounced after a few days, and this gelling further stabilizes a deposit (4) at a solids content of about 25 pounds per cubic feet or 85 percent water, by volume. On the other hand, if the shear stress on the bed applied by currents or wave action later exceeds the shear strength of the higher order deposit, the aggregates can quickly be suspended, and the lower order suspended aggregates are more durable than the bed surface and can survive intact during transport over long distances. Further, a suspended aggregate is free to rotate, and a velocity gradient will impose a maximum shear stress on it one-eighth of that on a fixed bed. Aggregates can participate in repeated suspensions and depositions, and such suspended aggregates can deposit, or form higher order aggregates and deposit at a greater rate. The importance of these processes will be apparent in a later description of the overall patterns of sediment movement in South Bay.

It is apparent that suspended materials are transported in the Bay system by the water flow.

Patterns of Sediment Movement in the Bay System

These fine sediments enter the Bay system as suspended material in tributary streams. Large amounts enter at the beginning of storm runoff, and the rate diminishes rapidly thereafter. Approximately 4.4 million tons enter the Bay system annually. Approximately 80 percent of the total sediment inflow enters with Central Valley drainage, and the remainder enters with tributary local stream flows. Most of the sediment enters during winter.

These suspended particles are dispersed and transported as individual mineral particles in the stream waters. They become cohesive in the region where ocean and stream waters mix. This mixing region moves oceanward during high freshwater flows, and recedes landward during lower flows. Their movement thereafter depends on the extent of their aggregation, which determines settling velocities, and on local hydraulic conditions. The overall pattern can be described as follows (5).

When the sediment-laden storm waters reach the broad expanses of the shallow bays in the system, where tidal velocities are low, the particles settle to the bed. Some also settle in portions of channels, turning basins, and harbors where velocities are insufficient to keep them in motion. The onshore breezes that occur daily during spring and summer months generate waves in the shallow areas which suspend the recently deposited materials and keep them in suspension while even slow tidal currents circulate sediments throughout the system. Measurements of particle size distribution like those shown in Figure 2 on samples taken from the surface of the bed in shallow areas of San Pablo Bay show changes from fine materials after high runoffs to increasingly coarser materials in summer (6). Studies of resuspension in shallow parts of San Pablo Bay using sediments labeled with a radioactive tracer showed their resuspension by wind-generated waves (7). The Bay waters are muddy during these regular windy periods and the greatest rate of shoaling in Mare Island Strait and in local harbors occurs during late spring and early summer as these sediments are circulated with the slow tidal currents and settle and accumulate in deep areas where neither wind-generated waves or tidal currents are sufficient to resuspend them.

Aggregation of suspended particles is facilitated by collisions with each other resulting from shearing due to wave and current interaction with the bed and due to the net movement of ocean water landward under the fresher outflow in regions of fresh- and ocean water mixing. This latter process causes recycling of suspended aggregates upstream near the bed until carried into the overlying, seaward flowing, fresher water by turbulent mixing, then downstream until in broad regions having low velocities where settling to the bed or to lower depths occurs. Particles can continue to recirculate and grow in such regions.

The Bay system is comprised of large shallow bays connected by narrow openings. This configuration causes lags in the progression of tide changes through the system. The dimensions are such that parts of the Bay system are resonant to the tides. The tide range at Alviso is greater than that at the Golden Gate because of resonance. These two processes cause circulation of waters from San Pablo Bay down into southern San Francisco Bay by tidal currents (8). South Bay fills from and empties to South San Francisco Bay with each tide through Dunbarton narrows, so that sediment can be carried into South Bay with tidal waters.

Winds have a major role in estuarial sediment transportation. As indicated above, wind-generated waves resuspend material deposited in the broad expanses of shallow water found in the Bay system and keep such material in suspension while tidal currents move the suspended material from one part of the system to another. Winds also produce additional circulations. Sustained winds drive water at the surface of the Bay downwind. A return current develops in waters below. Suspended sediment material is more concentrated near the bed, and these return currents move material suspended by waves up wind, often toward a channel. While these currents in shallow areas are slow, so are tidal currents in such areas. It is evident that movement of sediment into and out of South Bay through its connection to the broad shallow South San Francisco Bay and the circulation of sediment in South Bay depends on the direction, strength, and duration of winds.

Local Sediment Sources

Sources of the sediment that circulates in South Bay are the local tributary streams and South San Francisco Bay. There is very little information that shows the relative contributions of these sources, but the following will lead to useful conclusions.

Figure 2 presents the particle size distributions of the surface one centimeter of sediment in South Bay and the southern portion of South San Francisco Bay. Points labeled 5,6,7,8, and 9, are analyses of samples obtained from South Bay, with station 5 located near the center, and subsequent stations located along the channel toward and into Alviso Slough. The data plotted in Figure 2 shows that the surface of the bed was sandy in the fall of 1961 and early winter of 1962 up to at least March 1. Samples collected 1-3 May had a greatly increased clay content, however. As described above, resuspension by waves occurs during summer. Such resuspension preferentially removes finer particles and leaves the coarser. The large increase in clay shown in Figure 2 indicates that significant deposition of new material occurred between March 1 and May 1.

Flows and sediment production of tributary streams during the 1962 water year are included in the Appendix. These data show that major local storm flows occurred in mid February before the March 1 samples were taken and lesser flows during March and April. Figure 2 shows only a small increase in clay content between October and March. Between March 1 and May 1, however, both clay and silt contents of the sediment bed surface increased markedly.

Sediment production in the Sacramento and San Joaquin Rivers was highest during February and early March, 1962. Transport into South San Francisco Bay by the tidal currents, with repeated deposition in shallow parts of San Pablo and San Francisco Bays and resuspension when the wind blew, would continue over many tidal cycles and would account for the delay in appearance in South Bay. Clay and silt would be particularly favored in such transport and would further account for the unusual change in texture.

These considerations only show that input of sediment containing predominantly clay and silt is largely through Dunbarton narrows. The sandy portion of the bed material that remains in fall very probably originated from local streams because tidal currents in this portion of the system have limited competence for transporting sand. The streams also contribute some clays and silts. Clay and silt sediments appear to be the material causing problems in drainage channels and marinas, and for planning purposes it is useful to note the significance of clay and silt input through Dunbarton narrows.

The depth of the channel through Dunbarton narrows averages about 35 feet below mean lower low water, and the channel is about 2400 feet wide. Tidal currents range up to 2.0 knots during both flood and ebb (9). The magnitude of the flow can be calculated, using a conservative average current of 1.0 knot, to be about 140,000 cfs. Such flows into South Bay occur approximately half of the time, every day. From experience in other parts of the Bay system the concentration of suspended sediment in the flows can be several hundred parts per million during windy periods. Comparison of these discharge and sediment concentration figures with those of local streams shows that vastly greater amounts of material can be supplied to South Bay by tidal flows through Dunbarton narrows than by local streams.

Local Water and Sediment Circulation

Tidal currents in South Bay result from flows through Dunbarton narrows caused by the continually raising or lowering water surface in San Francisco Bay. Water flowing into South Bay on a rising tide through the narrows fills the broad areas of South Bay which in turn causes flow into the sloughs. The delay caused by the time required to raise the water level in South Bay results in a lag between the maximum in flood currents at Dunbarton narrows and in Coyote Creek of about 45 minutes. A similar lag can be observed between the times of slack water at the end of flood. The basin empties almost uniformly, however (10).

The currents in South Bay are determined by the bed configuration. A copy of the appropriate portion of the U.S. Coast & Geodetic Survey Chart 5531 is presented in Figure 3. This figure shows a main channel extending from Dunbarton narrows to Coyote Creek in a location that leaves about two-thirds of the shallow mud flat area southwest of the channel. This asymmetric location of the channel, together with the auxilliary channels of the Palo Alto side, causes a slow clockwise circulation over these mud flats. (It might be argued that the auxilliary channels formed or remained open as the bay filled because of the clockwise circulation in this area.)

Tidal currents in the main channel, where strongest currents occur, range up to 2.0 knots, or 3.4 ft/sec, near Dunbarton Point and peak currents diminish rapidly up the channel from the narrows to about 1 ft/sec in Coyote Creek. Tidal currents in the shallow areas are a few tenths of a foot per second at the strengths of flood or ebb.

These slow currents are very effective in moving sediment materials that are kept in suspension by wind-generated waves. The shear stress on the bed surface imposed by waves is greater for a given wave in shallower water. Deeper areas of the bottom experience less stress, and resuspension is not so rapid. The marked uniformity of the shallow mud flats result

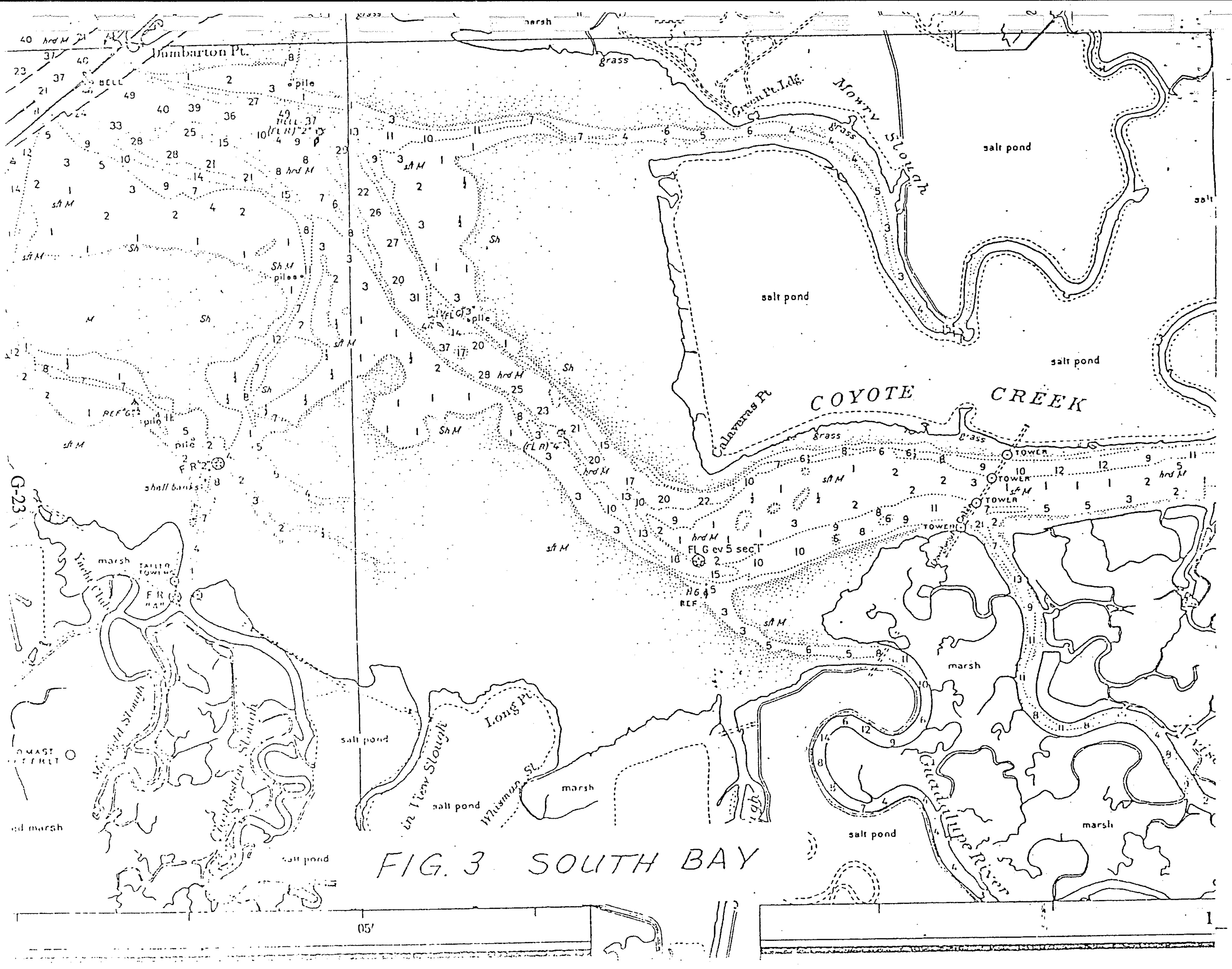


FIG. 3 SOUTH BAY

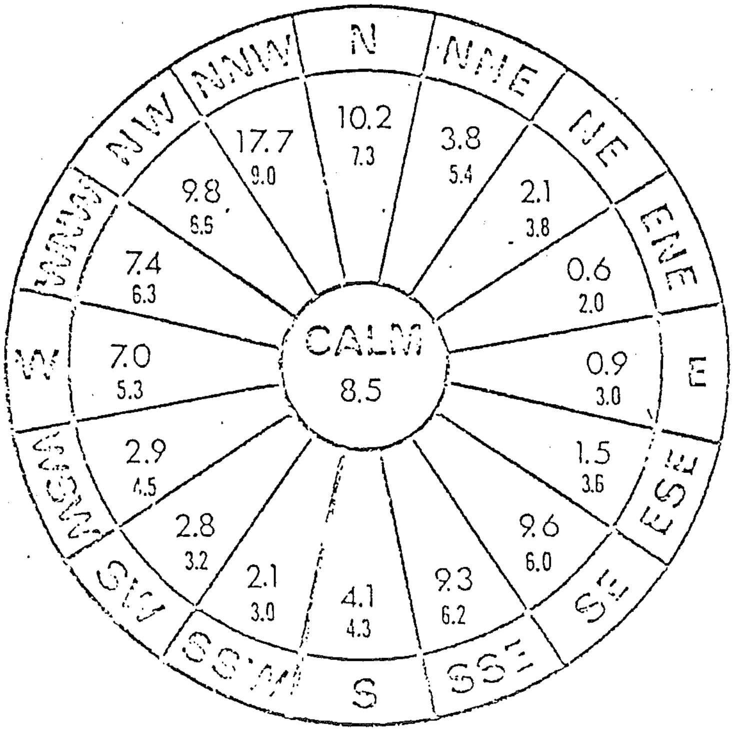
with only drainage channels, maintained by the currents as the area drains, marring the otherwise flat bed.

Winds that generate waves in the South Bay develop down-wind surface currents, as described, and a wind set-up or super-elevation of the water surface at the downwind shore. It is this super-elevation that causes the upwind current near the bed.

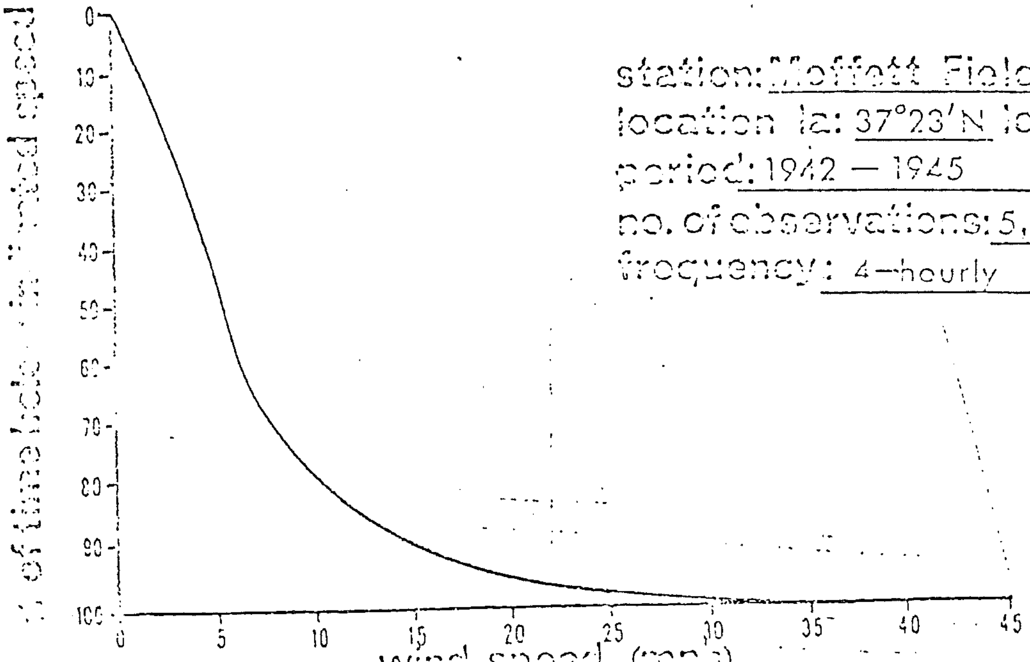
It is evident that winds are a major factor in the local circulation of sediments. Average winds at the San Jose Airport and at Moffitt Field are presented in Figures 4 and 5 (11). These figures show that the stronger and most often occurring winds come from the northwesterly directions down the axis of the Bay. The long fetch, or distance of exposed water up wind, makes winds from the northwest especially effective for generating waves.

It is evident from this qualitative description that fine sediment is brought into South Bay through Dunbarton narrows and is repeatedly suspended, circulated throughout the area, and deposited whenever or wherever the wave action subsides and the currents are slow. Typical concentrations of suspended sediments during windy periods are about one gram per liter. A large amount of suspended sediment is circulated, and any region sheltered from winds and waves or strong currents that receives water by tidal flow from the mud flat areas will shoal rapidly. Such shoaling would be most rapid in the spring and early summer months.

The Baylands, formed from past sedimentation and the establishment of marsh plants, are nearly level. Except during storms the flows in the stream mouths are tidal flows, and because the slough channels that comprise the stream mouths widen toward their ends, the competences of the flows to carry coarse materials from the land diminish as the termini are approached. In the various streams, therefore, the sediments in the sloughs that cross the baylands will be largely Bay mud with lenses of fine sand deposited from the streams during ephemeral storm flows. The stream beds above the ends of tidal flows will be found to be composed of progressively coarser material upstream as the slope increases.

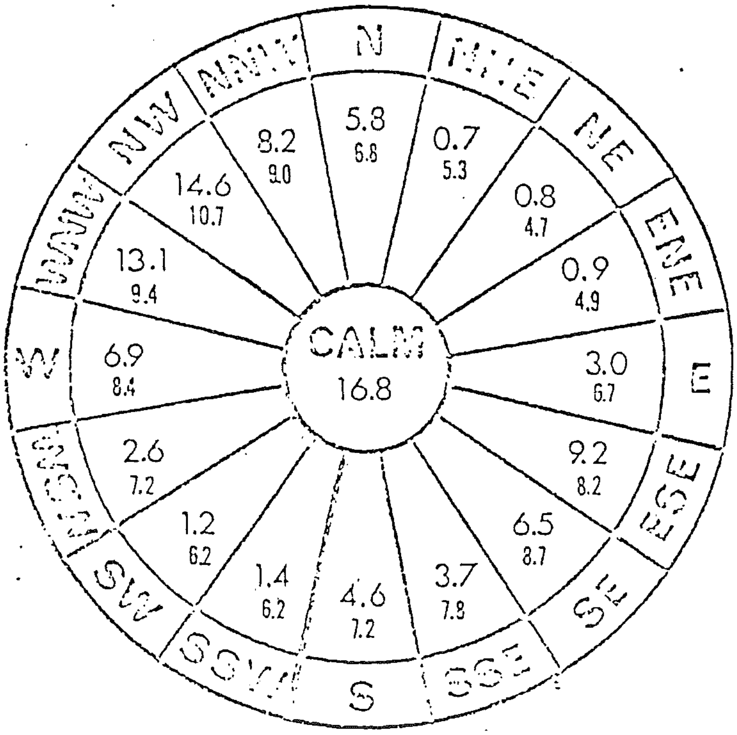


percentage distribution of wind directions
with mean wind speed beneath



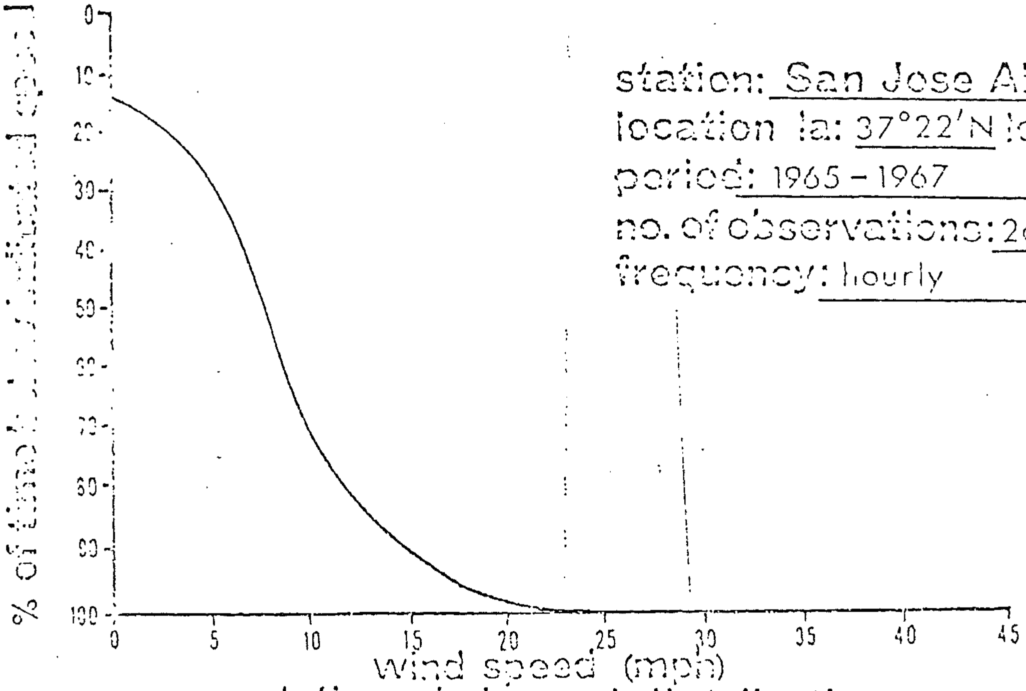
station: Moffett Field
location lat: 37°23′N lon: 122°04′ W
period: 1942 — 1945
no. of observations: 5,477
frequency: 4-hourly

cumulative wind speed distribution



percentage distribution of wind directions with mean wind speed beneath

station: San Jose AP
location la: 37°22′N lo: 121° 55′W
period: 1965 – 1967
no. of observations: 26, 276
frequency: hourly



cumulative wind speed distribution

FIGURE 5 WINDS AT THE SAN JOSE AIRPORT

SELECTION OF A PLAN FOR MINIMUM SHOALING OF BAY MUD

Three alternative Baylands flood control dike plans have been suggested by the study group:

The Inboard Dike Plan
The Channel Diversion Plan
The Outboard Dike Plan

These plans are presented elsewhere in the report. From the description of sediment transportation processes given herein it is evident that any modification that requires deepening the portions of channels that are subject to tidal flows will create a facility that will require periodic maintenance dredging. It has been reported that maintenance dredging is necessary now to provide adequate capacity for storm flows.

Accumulation of Bay mud that is carried into the channel from South Bay by tidal currents will occur during the major portion of the year when there is little or no stream flow (see Appendix A). Deposition will occur most rapidly near the sides of the channel where currents are slow and where the rate of aggregation is enhanced by flow separations at irregularities of the dike and at piers or other structures. The rate of resuspension by wind will be reduced in the lee of dikes, and resuspension by tidal currents will be limited wherever the currents are slow, such as in the corners of the channel and on the insides of bends. Accumulation of mud will therefore occur most rapidly along the sides of a channel and in the neighborhood of structures and bends.

A cross section will develop if the sediment is not removed that includes a narrow channel, which is maintained by tidal flows, and shallow mud flats that make up the remainder of the section. These deposits will stabilize. Submerged deposits gel, as described above, to a deposit that is 80 to 90 percent water by volume. When the deposition, usually along the sides of a channel, progresses until mud is exposed for significant periods during times of low water, the surface mud dries and the internal structure of the deposit irreversibly alters to a more dense arrangement. Large cracks develop in the mud surface as the material shrinks in volume, and the strength of the material increases. Further deposition and partial water loss eventually provides conditions suitable for the growth of plants, and the deposit is firmly stabilized against existing normal flow conditions. All stages of these filling and stabilizing processes can be observed in existing Baylands drainage channels.

The Channel Diversion Plan appears to be the plan that will require the least maintenance dredging for removal of Bay mud because it combines the flows and discharges them through Alviso Slough. Mud from South Bay accumulates in all deepened storm drainage channels through the Baylands during the major portion of the year that stream flows are small or negligible. The smaller area of maintained channel bottom in the Channel Diversion Plan would require the removal of a smaller total volume of sediment. Some economy would also be obtained from the reduced moving time for maintenance dredges that would result from use of one channel. Additional advantage could be obtained by straightening Alviso Slough, and the channel cross section should be made as narrow for the design storm flow as available dike elevation will allow, with the channel depth close to or less than that maintained by tidal flows.

Alviso Slough has the advantage of being contained by marshland almost to the Coyote Creek channel without an entrance channel across mud flats as has Guadalupe River. The suspended sediment concentration in the Alviso Slough water would therefore be less during windy periods which would favor a lower shoaling rate. Whether combining the storm-water flows and discharging them through Alviso Slough is the best plan from the cost standpoint, however, depends on the cost of the construction as well as maintenance. Preliminary designs of the plans should be made and equivalent annual costs of construction and maintenance compared.

Several approaches to evaluating the reduction in shoaling that would be expected from a channel modification are possible. A comparison of bottom surface areas for the channels before and after modification would give a first estimate. The volume of material deposited per unit of time is approximately proportional to the bottom surface area. Less approximate evaluations can be made by means of a physical or a mathematical model. The sources of sediment materials and the transportation processes described, together with the alternate plans proposed, lead to the following recommendations.

Recommendations

These recommendations are presented for the purpose of selecting a Baylands flood control dike plan for minimum shoaling and maintenance costs:

1. Design diked channels according to the Plans that will contain the storm flows selected for design when a winter spring tide prevails in South Bay. The channel depth should be comparable to existing water depths or less and the dikes should be as close together as allowed by upstream water surface elevations and dike construction requirements. The channels should be hydraulically clean and free of sharp bends.

2. Estimate the costs of the dikes and the benefits in addition to flood control that might result from the several plans.

3. For a first estimate calculate the area of channel bottom covered by tidal waters and determine the relative costs of maintenance dredging from the costs of the area presently dredged.

4. Estimate more precisely the rate of shoaling in the most favorable configuration, using a mathematical model, if necessary to determine the cost of the project for a build or not build decision.

5. Large amounts of sediment move in suspension in South Bay. Any increase in water depths beyond those that now sustain themselves, such as deepened navigation channels and mooring areas, and many structures that shelter the water surface from winds or impede the flow will enhance deposition rates and reduce resuspension rates: Shoaling will occur and continuing maintenance dredging will be required if water depths are important. Any modification to the bed and shoreline should be made in ways that minimize undesirable shoaling.

These recommendations should provide a means for making a desirable selection of a Baylands plan with the limited knowledge of the system available at this time.

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NATIONAL SYMPOSIUM ON URBAN HYDROLOGY, HYDRAULICS,
AND SEDIMENT CONTROL
(University of Kentucky, Lexington, Kentucky,
July 26-29, 1976)

SANTA CLARA VALLEY WATER DISTRICT
URBAN HYDROLOGY
USE OF THE CORPS OF ENGINEERS FLOOD HYDROGRAPH PACKAGE HEC-1 AS A PREDICTIVE MODEL

by

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Abstract. The effect of urbanization on floods has been successfully and uniquely modeled. The model couples concepts that simplify the urban hydrologic system, without loss of appropriate response, with an existing nonproprietary computer program. It is also easy and inexpensive to utilize and has given results consistent with statistical analysis of observed data. The concepts were developed and tested in the large metropolitan area of San Jose, California.

The first part of the paper develops a concept of subdividing an urban area into hydrologically equivalent pervious and impervious elements. For each of these elements, synthetic unit hydrograph parameters were calculated separately; and inflow hydrographs at storm drain system inlets were obtained.

The second part presents the storage-discharge relationships determined from storm drains, manholes, gutters, and streets. This storage-discharge relationship is regional in nature and applicable to urban areas of the type found in San Jose for the purpose of flood routing.

In general, the model accounts for the effects on runoff due to two major urban changes, increased imperviousness and channelization. It uses the existing HEC-1 computer program developed by the Corps of Engineers Hydrologic Engineering Center. This paper discusses the assumptions and the required inputs of the program. It does not discuss the theory or logic of the program itself; preknowledge of the program is assumed.

Introduction

The Santa Clara Valley Water District (SCVWD) is a special district with the dual responsibilities of flood control and water supply for Santa Clara County. Santa Clara County is located in the San Francisco Bay Area of California at the southern end of San Francisco Bay. It contains several cities, the largest being San Jose with a population of about 600,000.

At the turn of the century the population of Santa Clara County was not more than 100,000. The growth was relatively slow until the mid-fifties. At that time the population was only 400,000. Between the mid-fifties and the seventies, the population tripled, and it is expected to double again within the next 20 years.

Installation of separate storm drain and sewage systems accompanied the rapid urbanization of the valley floor. The design was based on the rational formula:

$$Q = CIA \qquad (1)$$

and the return period of the storm drain design flows ranged from three to 10 years.

Urban growth is concentrated on the valley floor which is about fifty miles long in a northwesterly, southeasterly direction. The width of the valley varies from 14 miles in the north to less than one mile at midpoint, then to about five miles in the southern part before widening again into a neighboring county.

The area has a Mediterranean climate with temperatures ranging from 35° to 60° F in winter, and 50° to 100°F during the summer. The mean annual rainfall ranges from 14 inches on the valley floor to 56 inches on top of the Santa Cruz Mountains, which form the western boundary of the valley. Precipitation comes primarily in the form of rain; snowfalls rarely occur.

Meeting its flood control responsibilities, the SCVWD had previously developed a hydrology methodology for determining design flood flows from unurbanized areas. It is based on a regional frequency analysis developed from regressing measurable watershed characteristics with the statistics of recorded flood events.

This paper presents the urban hydrology methodology, just developed and presently being used by the District for urban areas. It determines the amount of urban floodwaters conveyed to flood control channels in a time sequence. It is applicable as a planning and design tool. It can also aid in defining the volumes of water that are retained in an urban area due to the constriction of storm drains, levees or streets.

Hydrologic Engineering Center Program "Flood Hydrograph Package", HEC-1

A brief description of HEC-1 will help in the presentation of this paper. It is not intended in this description to present the HEC-1 program. We feel that thorough knowledge of the logic and theory of the program is required before attempting to apply this method.

Briefly, HEC-1 is capable of "performing five major types of flood hydrograph analysis. These five types are listed below:

1. Optimization of routing parameters;

2. Optimization of unit hydrograph and loss rate variables;

3. Generalized precipitation (rainfall or snowmelt), runoff routing and combining operations to simulate hydrologic response of a watershed and its stream network;

4. Stream system computations for specified precipitation-depth-area relationships for the entire watershed;

5. Specialized precipitation streamflow simulation relative to multiple floods for multiple plans of basin development and the economic analysis of flood damages."

Because of the program's ability to solve the needed hydrological problems, HEC-1 was utilized to generate pattern hydrographs from rural areas, balance the peaks and volumes against results obtained from regression and correlation studies, and route and combine hydrographs as the problem requires.

In order not to deviate radically from the traditional use of HEC-1 in rural areas, a similar technique for urban areas was utilized by considering the characteristics of the respective hydrologic elements.

Hydrologic Elements of Urban Areas

Urban areas are composed of pervious and impervious ground cover elements. The hydrologic and geomorphic characteristics of an urban area can, therefore, be defined, if the characteristics of each of these elements are defined.

If the pervious part is assumed to be hydrologically similar to a rural area within the hydrologic region, then Clark's synthetic unit hydrograph parameters and loss rate functions for this part could be easily determined by optimization. Also, because of the well-defined channels of the impervious part, the unit hydrograph parameters can be calculated utilizing the principles of hydraulics. The loss rate functions in this case can be correctly assumed to approach zero.

It is assumed by utilizing Clark's synthetic unit hydrograph, that the movement of water can be defined by a linear relationship between storage and discharge as long as the water runs over the ground surface. However, the storm drain and street system comprises a storage facility, the flow of water through which is defined by the non-linear relationships of storage to outflow and/or storage to inflow.

The following sections discuss these relationships and how they were modeled in detail.

Hydrology of Urban Areas

In general, urban hydrology deals with the effects due to two major changes in the watershed, namely:

1. Increased imperviousness
2. Channelization

The increase in imperviousness is caused by rooftops, streets, parking lots, driveways and other man-made impervious ground surface cover. Channelization includes the flow of water in gutters, storm drains, improved channels, etc. An increase in imperviousness tends to reduce the loss rates and increase the runoff volumes and peaks, while channelization may either increase or decrease the peaks. This last effect depends on the magnitude of the flood and the storage-discharge relationship of the various channels or storm drain systems that convey that flood.

To determine the effect of increased imperviousness, every urban watershed is subdivided into areas tributary to the major storm drains. These areas are then transformed into equivalent rectangular subareas where the length is equal to the length of an equivalent street as shown in Figure 1.

The ratio of imperviousness is then used to subdivide this equivalent subarea into pervious and impervious parts. The following paragraph describes the computation of the paths water follows through these parts.

Given the total urban area (A), the land use, the percentage of street area, and the width of an average street, the length of the transformed area (Ls) can be obtained as follows;

$$L_s = \frac{As}{Ws} \qquad (2)$$

Where A_s = Street area = A x percentage of the area in streets (Table 1)

W_s = Average street width

and

$$L_i = \frac{Ai - As}{2L_s} \text{ and } L_p = \frac{Ap}{2L_s} \qquad (3)(4)$$

Where L_i = Length of overland flow for the impervious area.

A_i = Impervious area from Table 1.

L_p = Length of overland flow for the pervious areas.

$A_p = A - Ai$

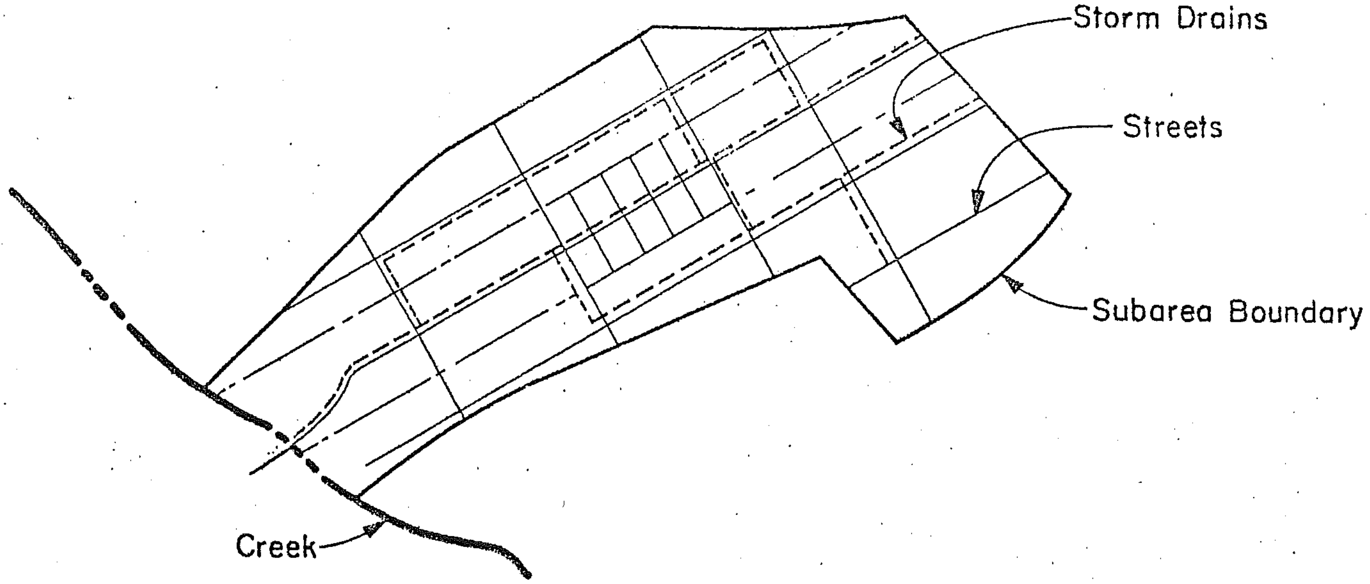
Table 1, which is used in the computations of the paths of water L_i and L_p, is based on the development practices of the City of San Jose and as determined by studying aerial photographs.

The flow hydrographs from both pervious and impervious parts are calculated at the inlets of the catch basins by employing the synthetic unit hydrograph procedure in the Corps of Engineers HEC-1 Program. For this purpose, separate time of concentrations of both the pervious and impervious parts are calculated as described in the next Section. The routing coefficients are calculated from the time of concentrations and the width of these areas. The loss rates, which are the third set of parameters needed for the computation of the synthetic hydrographs, are optimized for the pervious (rural) part and assumed as a constant value near zero for the impervious part. The waters generated from these parts are then combined, as needed, and assumed as inflow to a storage facility.

Table 1.* Ratio of Imperviousness and Street Area Percentages of Urban Areas, San Jose

No.	Land Use	Percent Impervious	Percentageof Street Areas to Total Area
1	Low Density Residential	25 - 45%	20 - 25%
2	Medium Density Residential	45 - 65%	20 - 25%
3	High Density Residential	65 - 85%	20 - 25%
4	Commercial	85 - 95%	20 - 25%
5	Industrial	85 - 95%	25 - 30%

*This table is applicable to flat land.



Schematic of Urban Subarea

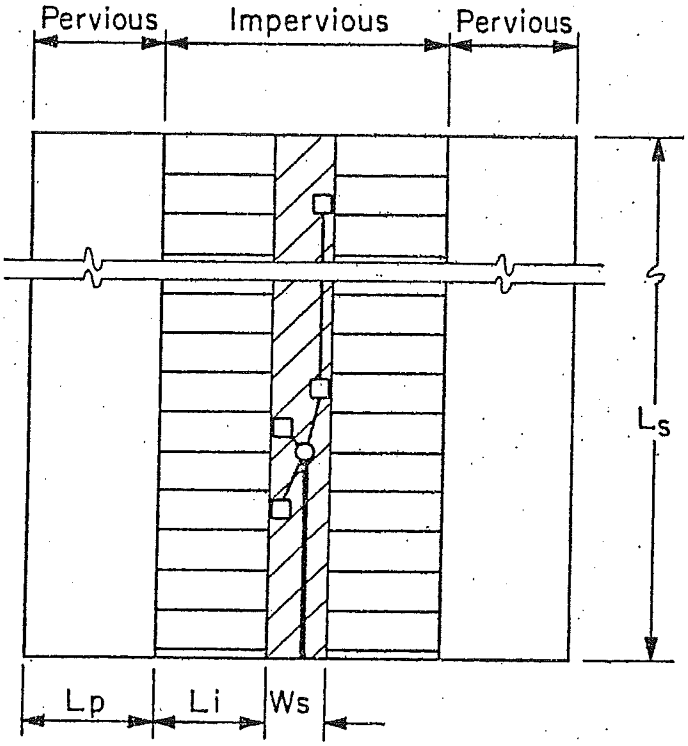


FIG. 1 Equivalent Urban Subarea

Because of the way the land, streets and flood control channels are constructed in Santa Clara County, most of the flood waters from urban areas are conveyed to the flood control channels by means of storm drain systems. The inflow hydrographs to these systems are then routed through the storm drain system utilizing the modified Puls routing method. For this purpose, storage-discharge curves are needed for the system of streets, gutters, manholes and storm drains.

Two actual urban areas (near Ross Creek, representing medium density residential and near Guadalupe River in downtown San Jose representing high density residential and commercial) were selected. Equivalent generalized drainage and storage plans (Fig. 2a and 2b) were developed from these areas. Storage-discharge curves were computed independently for each of the two study areas. The resultant curves were transformed to reflect unit areas, and then compared with each other. This comparison showed that the two curves were very similar. Therefore, an average smooth curve (Fig. 3) was developed and used.

Apparently, the storage-discharge relationship of the storm drain systems in the areas studied is not significantly affected by the density of development or the ratio of imperviousness. It is rather a function of the general slope of the ground surface and the storm drain design capacity. Because most of the urban areas in Santa Clara Valley are relatively flat, and similar storm drain design criteria was used throughout, it was assumed that the storage-discharge curve that was obtained from the above study is applicable to all urban areas in the Valley. It is advisable, though, to develop storage-discharge curves that characterize other urban areas if this method is to be used for areas different than Santa Clara Valley.

The assumptions that were made to facilitate the hydraulic computations of the storage-discharge curves are listed below.

a. The flow rate of every tributary to a main storm drain is equal to the total flow rate of the main storm drain system multiplied by the ratio of the tributary area to the total area.

$$Q_{trib} = Q_{total} \times \frac{A\ trib}{A\ total} \qquad (5)$$

b. The inflow to every manhole is equal to the tributary flow rate divided by the number of manholes in the tributary area.

c. It is assumed that there are three catch basins and six gutters to every manhole.

d. The slope of gutters and storm drains is assumed equal to the average slope of the ground surface.

e. A free flow at the outlet.

As shown in the Ross Creek case study (Appendix A), floods that have a flow rate larger than the storm drain capacity will be temporarily retained generating local flooding in low spots, street crossings, behind levees, etc. For large floods, this process tends to attenuate the local flood peaks generated from urban areas and reduces the effect of urbanization on flood flow peaks within the main flood control facilities. Because of this attenuation, the assumed and optimized loss rates in the urban areas have little effect on the peaks of routed local hydrographs during large floods.

Computations of the Synthetic Unit Hydrograph Parameters

Clark's synthetic unit hydrograph parameters were determined by taking actual subareas and determining the width of the impervious and pervious parts (L_i, L_p) as described above together with immediate gutter length (Equation 8). Izzard's overland flow formula was used to obtain the time of concentration through L_i & L_p, while the time of travel through the immediate gutter (distance between the furthest lot and nearest manhole) was hydraulically determined by assuming average velocities.

The time of concentration for each sub-subarea was computed as follows:

a. Time of concentration of impervious sub-subareas:

1. Overland flow travel time, T_o (from IZZARD's formula):

$$T_o = \frac{41\ c\ Li^{1/3}}{Si^{1/3}\ (CI)^{2/3}} \qquad (6)$$

Where c = a retardance coefficient (Table 2)

Li = Length of impervious area in feet ($Li = \frac{Ai-As}{aLs}$, Fig. 1)

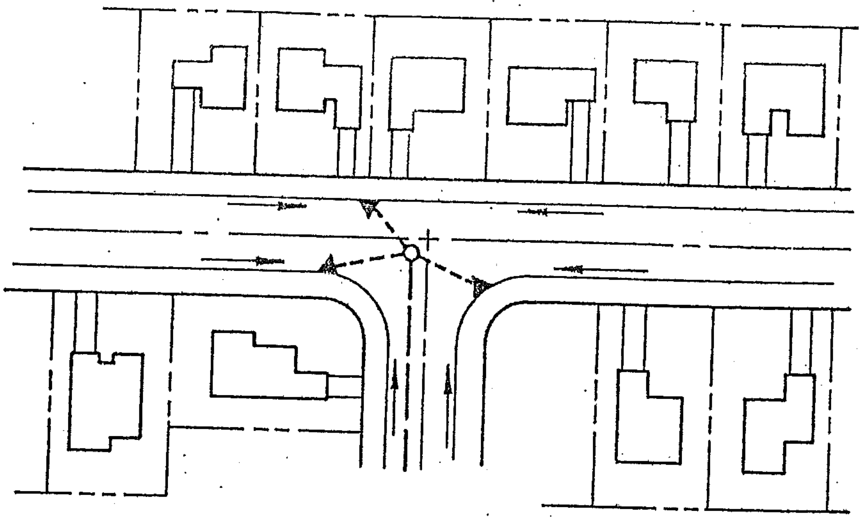
Si = Slope of impervious area

C = Runoff coefficient, (Table 3)

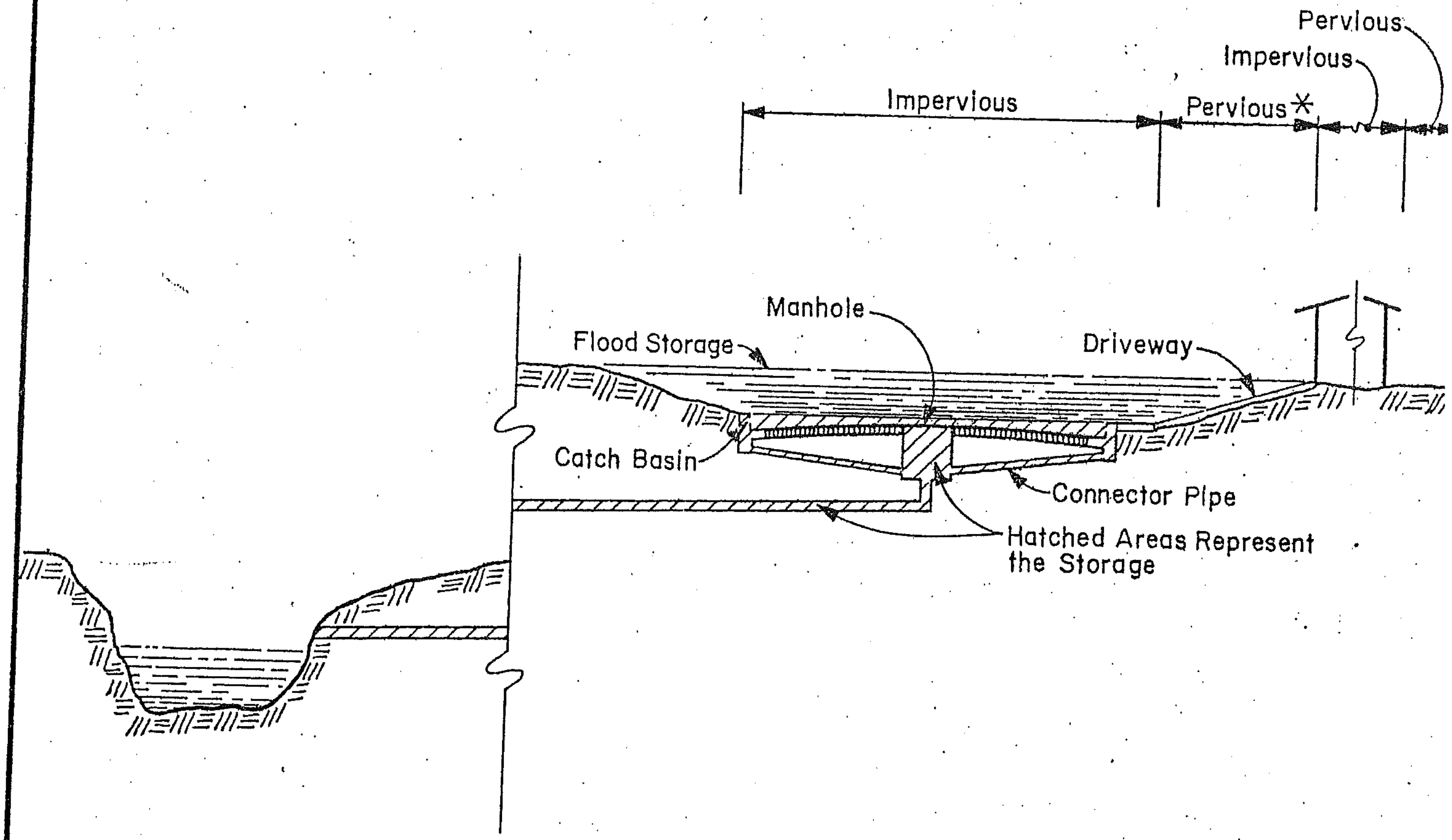
I = Rainfall Intensity

2. Immediate Gutter travel time, T_g

$$Tg = \frac{Lg}{Vg} \qquad (7)$$



(Fig. 2a.) TYPICAL LAND USE AND DRAINAGE PLAN



(Fig.2b.) THE GENERALIZED STORAGE FACILITY OF URBAN SUBAREAS

*Driveways and Sidewalks are included in the Impervious Portion of the Equivalent Urban Subarea.

STORAGE-DISCHARGE CURVE FOR 1 SQ. MI. OF URBAN AREA

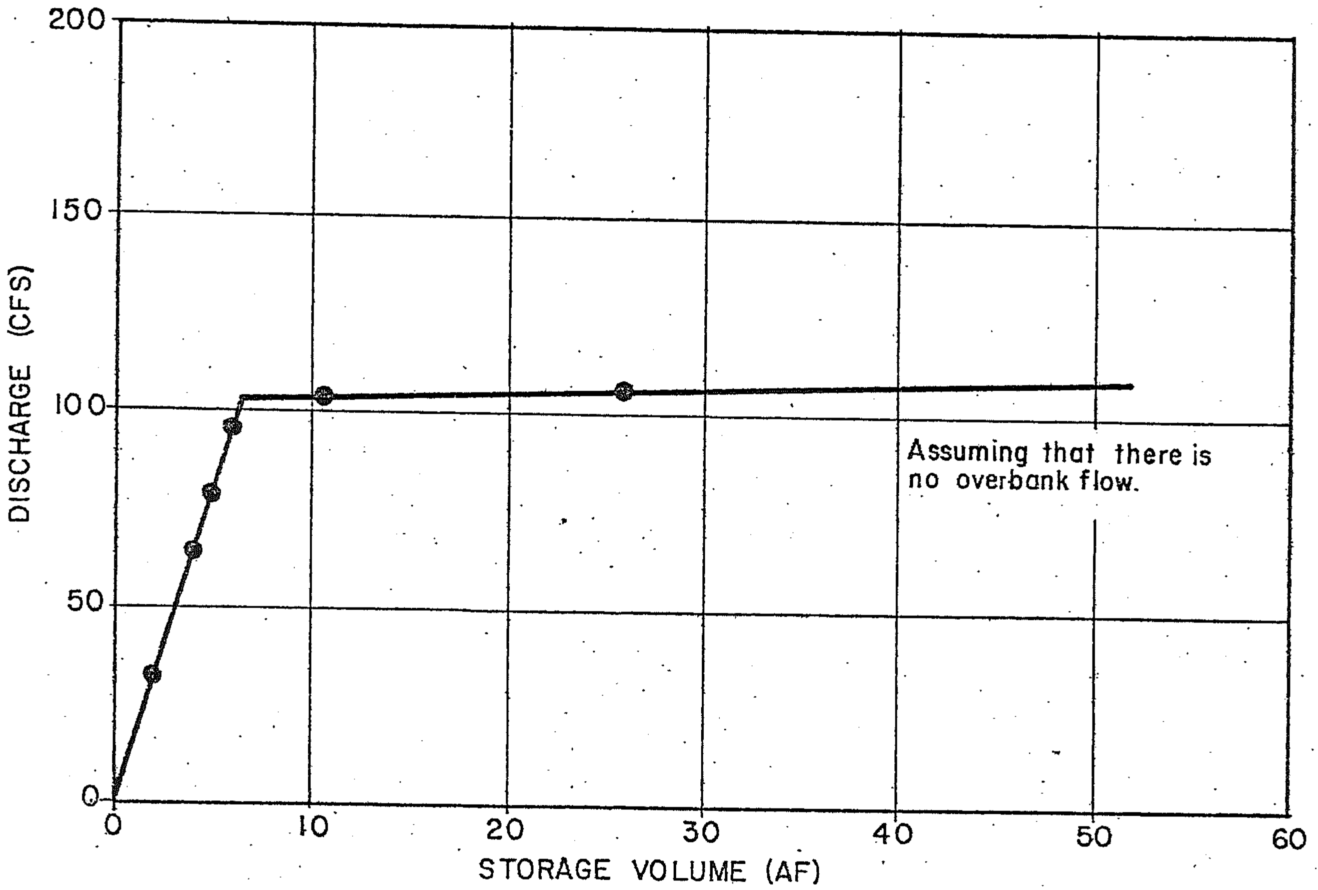


Figure 3

Table 2.* Retardance Coefficient c

Type of Surface	Value of c
Asphalt	.007
Concrete	.012
Tar and Gravel	.017
Lawns	.046
Dense Bluegrass Turf	.06

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*Adapted from IZZARD's.

Table 3.* Runoff Coefficients for Urban Area

Class of Area	Limits of Slope	Coefficient
Streets	-	
Asphalt	-	.70 - .95
Concrete	-	.80 - .95
Driveways	-	.75 - .85
Roofs	-	.75 - .95
Lawns, Parks, Etc.		
Sandy Soil	S < 2%	.05 - .10
Sandy Soil	2% < S < 7%	.10 - .15
Sandy Soil	S < 7%	.15 - .20
Heavy Soil	S < 2%	.13 - .17
Heavy Soil	2% < S < 7%	.18 - .22
Heavy Soil	S < 7%	.25 - .35

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*Design and Construction of sanitary and storm sewers, ASCE Manual of Eng., Practice No. 37, 1960

Where Lg = Immediate gutter length

Vg = Average flow velocity from Manning's equation.

The computation of the immediate gutter length was obtained by studying the two areas mentioned adjacent to Ross Creek and the Guadalupe River. From these areas, nine representative subareas were chosen. The immediate gutter length, which is the maximum path of water from the furthest lot to the first manhole, was regressed against the storm drain density. The storm drain density is defined as the total length of storm drains in miles divided by the total urban subarea in square miles.

The regression analysis generated the following linear relationship.

$$Lg = 4000 - 470\ S_d \qquad (8)$$

Where, Lg = is the immediate gutter length which must be equal to, or greater than 300 feet

S_d = is the storm drain density in

$$\frac{mi}{mi^2}$$

The multiple determination coefficient for this relationship is $r^2 = 0.66$ for the intercept and 20% ± for the slope.

The total time of concentration of an impervious sub-subarea is then equal to:

$$Ti = To + Tg \qquad (9)$$

Thus To and Tg can be considered respectively as an approximate measure of the land use as it relates to ground cover, and a measure of the distribution of the storm drain system within the urban area.

b. Time of concentration of the pervious part (T_p)

$$T_p = T_i + T_r \qquad (10)$$

Where, $Tr = \dfrac{41 \times c \times L_p^{1/3}}{S_p^{1/3} (CI)^{2/3}} \qquad (11)$

Tr is the travel time of overland flow through the pervious part.

$L_p = \dfrac{Ap}{2Ls}$ Length of pervious area (Fig. 1)

c, C, Sp and I are respectively, the retardance coefficient, the runoff factor, the slope and rainfall intensity.

c. Clark's routing coefficient (R) is computed taking into consideration the length and width of the sub-subareas. This value can be refined further by studying the generated hydrograph of similar hydrographs in the area. This modification is done based on the computation of the Muskingum (k) when X = 0.

Alternative Storage-Discharge Curve

Flood waters from urban areas in the San Jose area are conveyed to flood control facilities primarily by means of storm drains rather than by overbanking directly into flood control facilities. However, if the streets are designed to convey the excess water, above the storm drain's capacities, to flood control facilities at crossings and directly over bank, a different storage-discharge curve would result. To develop this curve, the average number of streets that cross the creek, per square mile of representative urban drainage area is calculated. The average slope of streets in the unit area is also calculated. Because most of the urban areas in Santa Clara County are relatively flat, this slope is considered to be the slope of an equivalent street. A storage-discharge curve is then computed for this equivalent street. This curve is then used to modify the storage-discharge curve that was obtained for the storm drains and is used when overbanking conditions exist (Fig. 4).

Summary of Application of Methods

To summarize, the following steps are needed in order to analyze an urban area. A general flow chart is presented in Appendix B.

a. Delineate urban areas tributary to flood control facilities.

b. Delineate subareas relative to the major storm drains.

c. Define the land use of every subarea and subdivide it into equivalent pervious and impervious sub-subareas.

d. Find the required total design storm in inches that has the desired duration and frequency of occurrence, and distribute it as desired. The Corps of Engineers' standard storm distribution has been utilized and appears adequate.

e. Determine the length of overland flows L_p and L_i for both pervious and impervious sub-subareas.

f. Determine the time of concentration for each sub-subarea by studying the overland flow lengths and storm drain densities.

g. Determine the routing coefficient 'R' for both sub-subareas.

h. Determine the loss rates for pervious areas by optimizing gaged rural watersheds of similar characteristics. Loss rates for impervious parts are assumed near zero.

i. Generate separate unit hydrographs and compute the respective pervious and impervious inflow hydrographs.

j. Define the type of storm drains and street pattern and determine the type of the storage-discharge function.

k. Combine the hydrographs generated in (i) as the problem needs, and rout the resultant combined hydrograph through the system determined in (j).

l. Combine the resultant hydrograph from (k) with the hydrograph traveling from the upper watershed, and rerout the total hydrographs.

m. Repeat the process form a to l, while moving downstream, to the point of interest.

n. Compare the results with recorded data if any is available.

Results

This method was applied on Ross Creek. The mathematics and the detailed analysis is outlined in Appendix A.

It was proven in the case study as well as in applying this method on other areas in the Valley that the characteristics of the frequency curve of recorded data can be reproduced.

STORAGE-DISCHARGE CURVE
FOR
1 SQ. MI. OF URBAN AREA

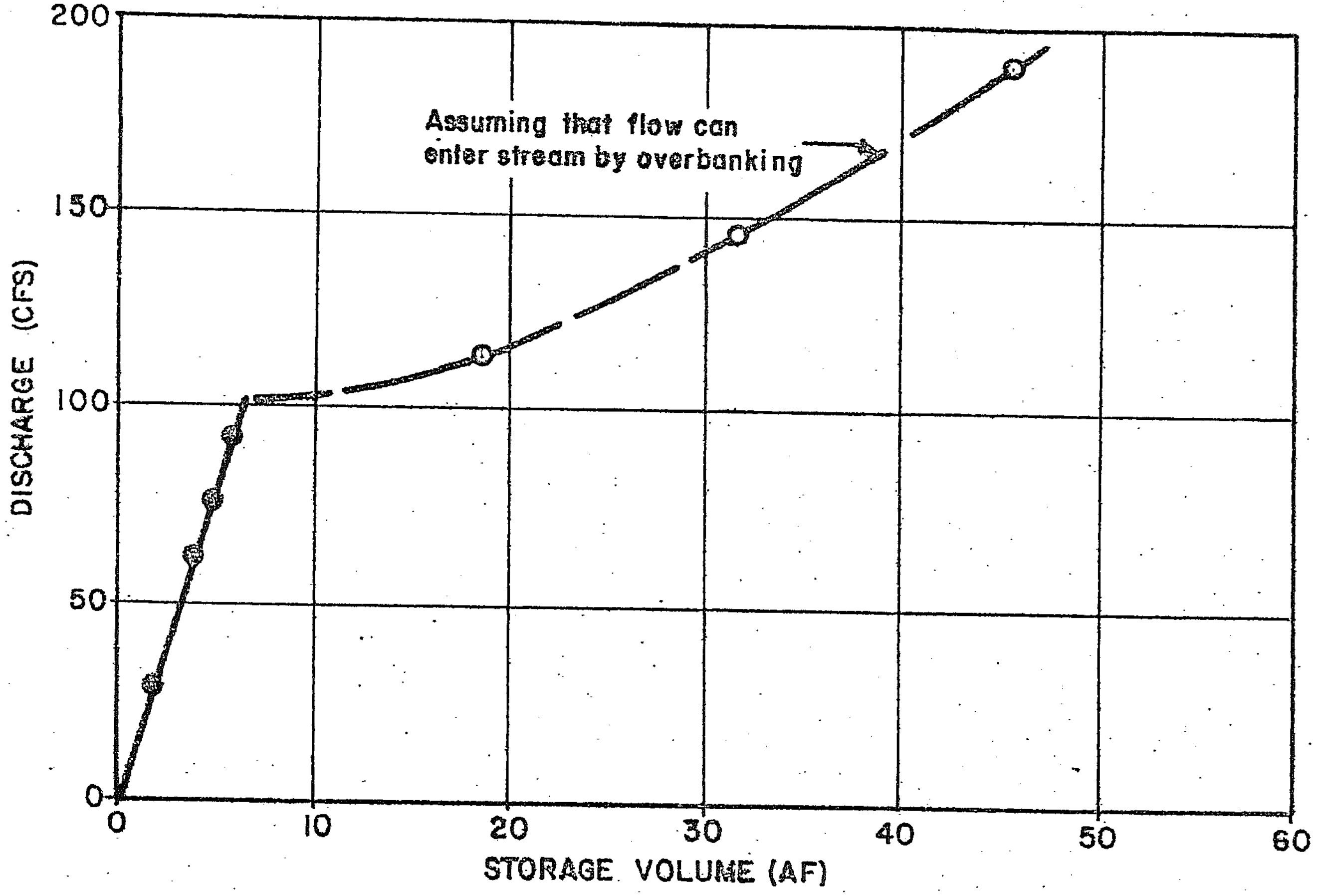
Assuming that flow can enter stream by overbanking

Figure 4

Acknowledgements

This study is indebted to the tremendous help and encouragement that was extended to us by the Chief Engineer Lloyd C. Fowler and Advanced Planning Manager Dave Gill. Without their support and backing, this work could not have been realized.

Mr. Randall Talley and W. James Sanchez are commended herein for their efforts in compiling and analyzing the data and testing the results.

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APPENDIX A. ROSS CREEK HYDROLOGY
A CASE STUDY
MARCH 1976

APPLICATION OF S.C.V.W.D. URBAN HYDROLOGY METHOD TO ROSS CREEK

General

The Ross Creek Watershed was chosen as a test basin. It has a 10 square mile drainage area set against the southern foothills of the Santa Clara Valley. The uppermost 2.9 square mile foothill area remains rural while the lower flatland areas have been extensively urbanized, primarily with medium-density single-family residences. Mean Annual Precipitation (M.A.P.) varies from 15 inches at the outlet to 30 inches in the upper reaches of the basin, 6.8 miles upstream.

There are two stream gages on Ross Creek conveniently located so as to separate the rural and urban areas. Station 21 instruments the rural drainage and has continuous and reliable records since 1946. Station 51 measures flow from the entire basin and has fair records since 1957. Table A-1 summarizes the peak flow records of these stations. Figure A-1 shows the watershed and the location of the various stations.

Table A-1. Recorded Annual Peak Flows

Station #21 D.A. = 2.86 sq. mi. M.A.P. = 25 inches				Station #51 D.A. = 9.96 sq. mi. M.A.P. = 19.5 inches	
YEAR	DISCHARGE (CFS)	YEAR	DISCHARGE (CFS)	YEAR	DISCHARGE (CFS)
1956	840	1974	158	1968	1550
1967	600	1946	140	1967	1030
1968	570	1975	137	1971	952
1958	480	1951	133	1963	880
1960	440	1966	110	1965	850
1964	370	1949	110	1969	845
1971	369	1954	98	1964	820
1963	330	1961	90	1970	550
1959	305	1955	85	1958	480
1970	299	1957	78	1966	320
1973	233	1953	62	1972	285
1962	214	1972	46	1961	230
1952	209	1950	42	1857	115
1965	200	1947	21	1960	45
		1948	20		

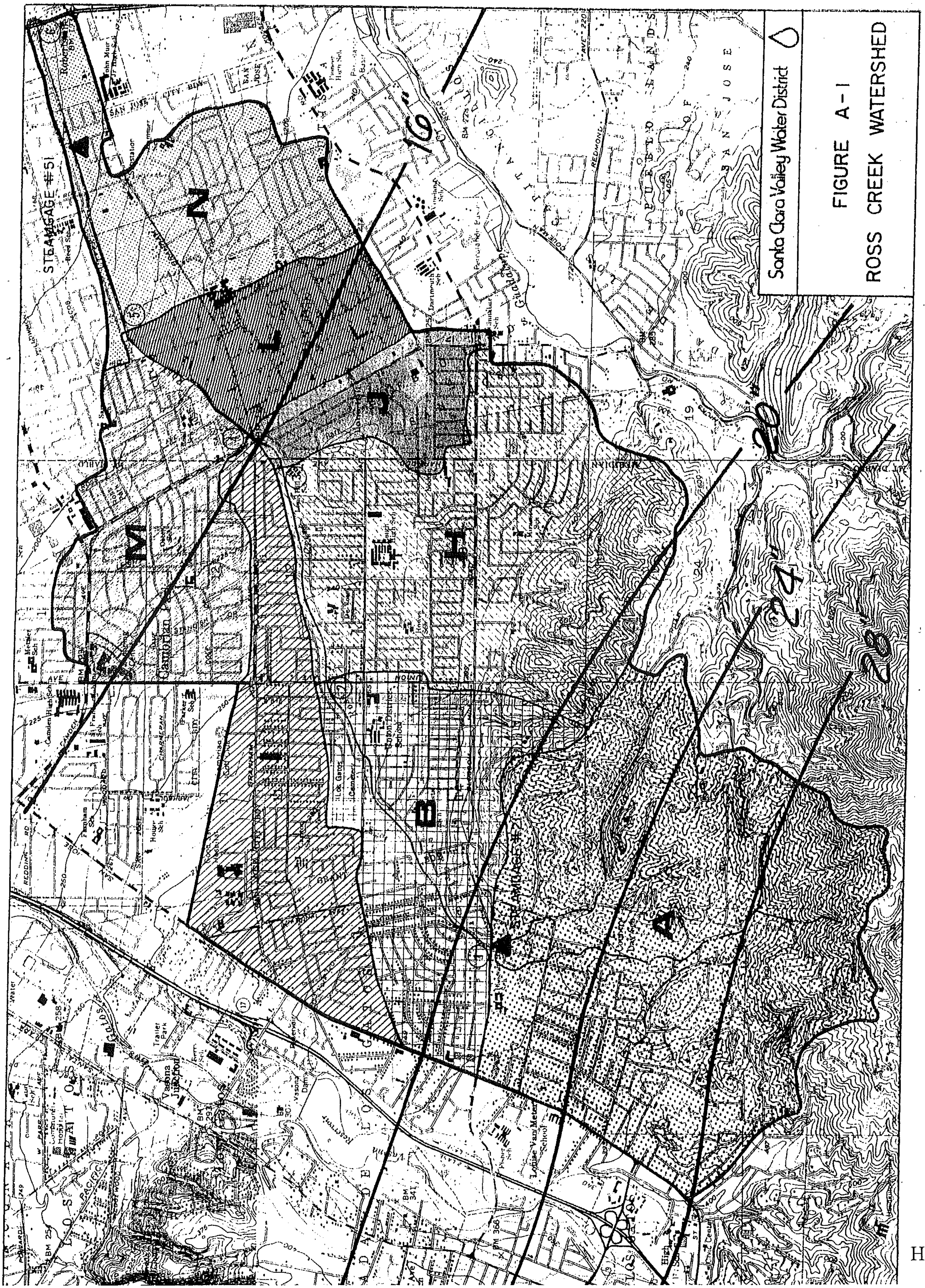

FIGURE A-1
ROSS CREEK WATERSHED

Frequency Analysis

Statistical analysis of this data produced the following results:

Station #21

$M = 2.202$

$\sigma = .4181$

$g = -0.42$

Station #51

$M = 2.686$

$\sigma = .3725$

$g = -1.47$

Using the Log Pearson Type III distribution the frequency curves were plotted on Figures A-2 and A-3 as a solid line. The dashed lines represent the frequency curves from the regional regression analysis. The regional regression equations developed by the District to define the statistics of hydrologic events for rural areas are:

Instantaneous Peak

$$M = \log (1.92575\ A^{0.78} P^{1.04}) \qquad \text{(A-1)}$$

$$\sigma = .4438 A^{0.03} P^{-0.018} \qquad \text{(A-2)}$$

1-Day Volume*

$$M = \log (.1808\ A^{0.923} P^{1.561}) \qquad \text{(A-3)}$$

$$\sigma = .9718\ A^{-0.007} P^{0.197} \qquad \text{(A-4)}$$

Where

A = Drainage Area (mi^2)

P = Mean annual basin Precipitation (in)

At Station 21, the frequency curves from both the regional and recorded data fit quite well, an evidence of the high applicability of the regression equations to that area (Fig. A-2). Using the results of these equations as an indication of what to expect if the entire basin had remained rural; a comparison with the statistics of Station 51 (Fig. A-3) show the effects of urbanization.

Curiously, the two frequency curves (regional and recorded) in Fig. A-3 are equal at about the 20% (5-year) frequency, a value sometimes used for urban storm drain design. The magnitudes of more frequent events are greater than that predicted from the regional regression equations while the magnitudes of the larger, more unusual events are less than expected.

An intuitive hypothesis would predict (as is generally accepted) that the urbanization of a basin would increase the volumes and peaks of more frequent floods relative to the flood control channel, while reducing the peaks of rare floods due to the constriction of the storm drains. If the hypothesis is true, (and the frequency curves at Station 51 now indicate that there is some basis for the hypothesis), then the objective of an urban hydrology should be to reproduce this characteristic.

Discussion and Sample Results

For application of the method, the ten square miles of area in the Ross Creek basin were subdivided into eight separate drainage subareas (subareas A, B, H, I, J, L, M, N) relative to the major storm drains (see Figure A-1). Each subarea, except subarea A, which is rural, was treated individually in determining the time of concentration and routing coefficient for the pervious and impervious parts. The hydrographs were then combined and routed through the storm drain system as per the developed procedure.

Typical procedural calculations are summarized for a sample urban area (I) of 1.11 square miles (see Fig. A-1). The area is medium-density single-family residential.

- Area (I)

  Total Area, A = 1.11 square miles

  Impervious Area, Ai = .63 square miles (60% of total area for medium density residential)

  Pervious Area, Ap = .48 square miles

- Overland Path Lengths

  Area of Streets = 25% of Total Area = .28 square miles

  Impervious, Li = 42 feet

  Pervious, Lp = 48 feet

- Time of Overland Flow

  Impervious, To = 15 minutes

  Pervious, Tr = 120 minutes

- Gutter Travel Time

  Storm drain density = 5.34 miles/mile^2

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*24-hour volumes are derived from the relationship $\log Q_{24} = .23 \log Q_{peak} + .77 \log Q_{daily}$. (U.S. Army Corps of Engineers)

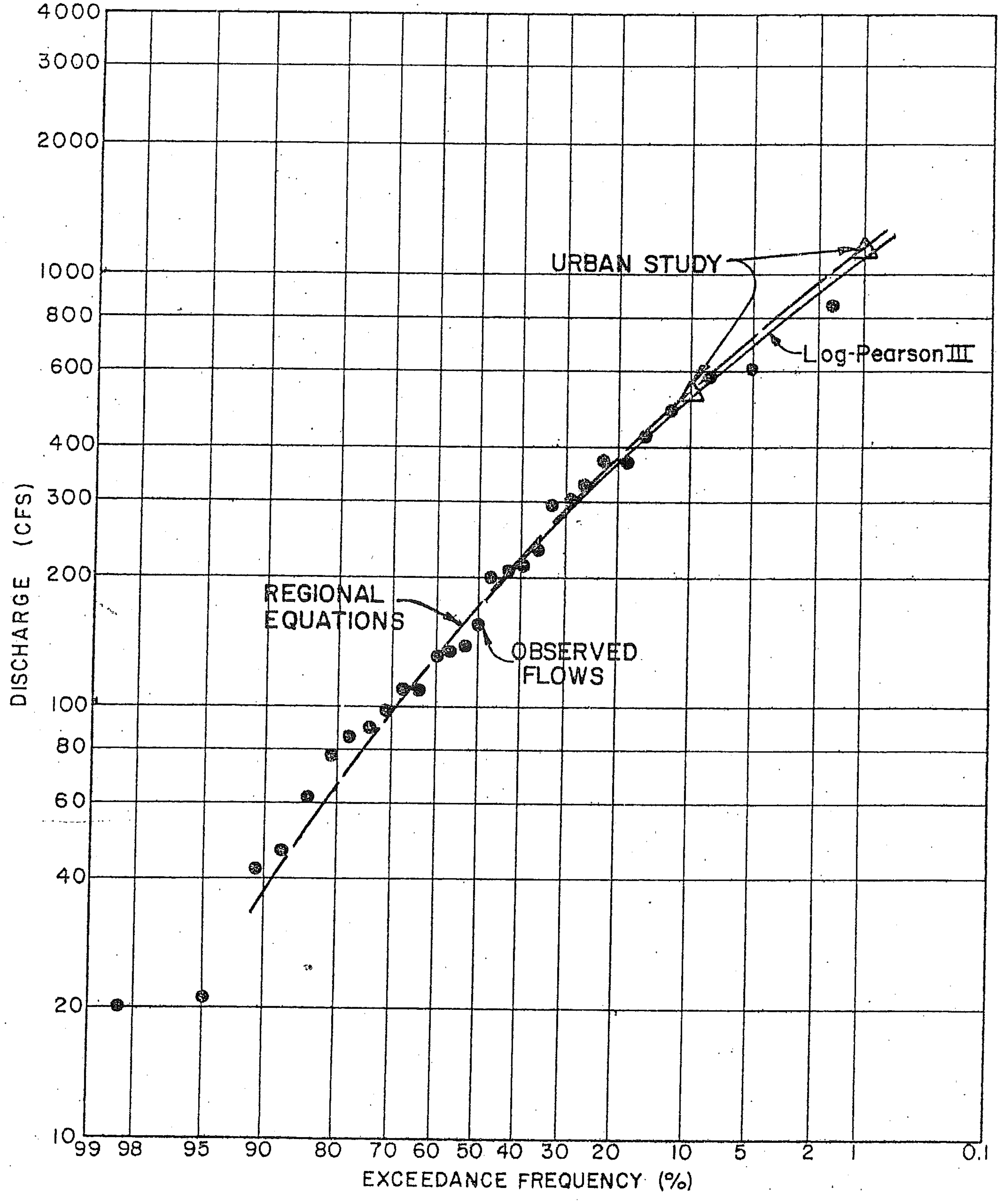


FIG. A-2 ROSS CREEK AT STREAMGAGE # 21

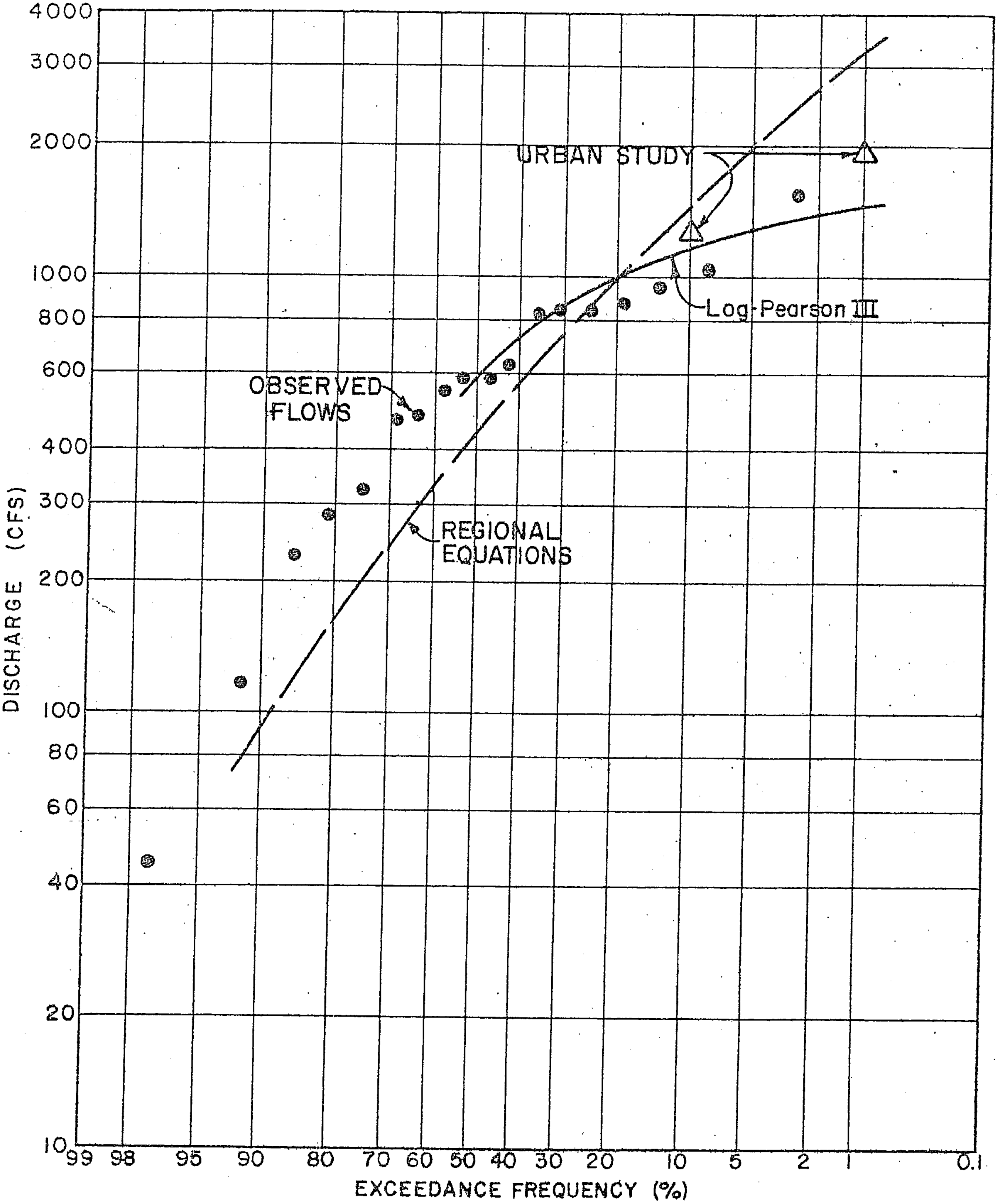


FIG. A-3 ROSS CREEK AT STREAMGAGE # 51

Length of immediate gutter path = 1,500 feet

Tg = 10 minutes

- Total Times of Concentration for Impervious and Pervious Areas

Ti = To + Tg = 25 minutes
Tp = Ti + Tr = 145 minutes

- Routing Coefficients, R

Impervious, R = 15 minutes

Pervious, R = 15 minutes

- Storage-Discharge Curve

The storage-discharge routing curve for the sample area was obtained by multiplying both ordinates of the standard unit curve (developed for a 1 square mile area Fig. 3 of text) by 1.11.

- Loss Rates

Table A-2 shows the loss rate functions. These values are obtained for the pervious part by optimizing rainfall and runoff historic events at Station 21. For the impervious part, low values were arbitrarily chosen.

Table A-2. Loss Rates

Loss Rates	Impervious Areas	Pervious Areas
1% Event	5%	30%
10% Event	10%	50%

The one percent and 10 percent flood frequency events were both tried. The final result of each frequency is plotted on Figures A-2 and A-3 and summarized in Table A-3.

The value for the 1% event at Station 51 obtained from use of this urban methodology lies between the value which would be expected from a natural drainage (as per the regional analysis), and the statistical analysis of recorded data by assuming a log Pearson Type III distribution. The application of Pearson Type III distribution is not entirely appropriate for this data because of the non-homogeniety generated from the gradual urbanization of the watershed. However, a graphical solution also makes it clear that the results of this urban method are substantiated by observations. The 24-hour volume also increased (7%) although not as significantly as in the 10% event (57%). The loss rates during a rare 1% event are already very low, so the increase in imperviousness due to urbanization has a lesser effect than in a 10% event where loss rates are much higher.

It could be easily shown that the hydrograph at the inlets to the storm drain system have increased in both peaks and volumes as a result of the changes in loss rates and time of concentration due to urbanization. The reduction in peaks at the storm drain outfall is a result of the temporary retention of flood waters in this storage facility. This temporary storage is illustrated for the sample area in Table A-4.

Conclusion

It can be concluded, therefore, that the statistical characteristics of the recorded data at Station 51 has been reproduced by this method and that the attenuation process of storm drains reduced the peak flow from urban areas relative to the main flood control channel.

Table A-3. Results of Study

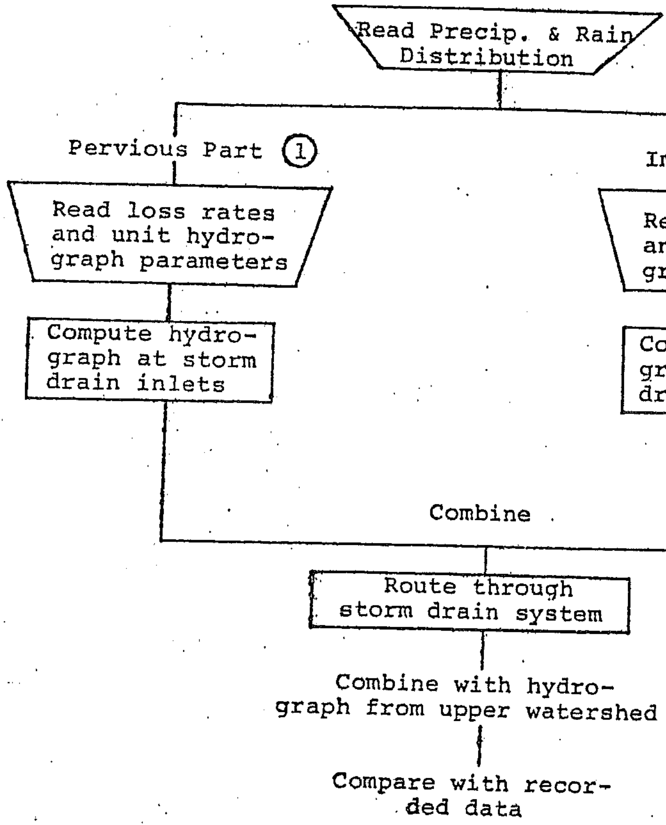
	Statistical	Regional Regression	Urban Hydrology Study
At Station #21			
1% Event Peak	1105	1140 cfs	
24-Hour Volume	-	941 cfs	
10% Event Peak	519	530	
24-Hour Volume	-	400 AF	
At Station 51			
1% Event Peak	1446	3240 cfs	1950 cfs
24-Hour Volume	-	2320 AF	2490 AF
10% Event Peak	1167	1460 cfs	1280 cfs
24-Hour Volume		960 AF	1510 AF

Table A-4. Volume of Waters in Retention (Sample Subarea-I)

Peak	Before Inlet 672 cfs	At Outfall 238 cfs	Volume in Storage
After 6-hours	224 AF	117 AF	107 AF
After 24-hours	539 AF	431 AF	108 AF
After 33-hours	546 AF	522 AF	24 AF

Appendix B

General Flow Chart


Read Precip. & Rain Distribution
Pervious Part ①
Impervious Part ②
Read loss rates and unit hydrograph parameters
Read loss rates and unit hydrograph parameters
Compute hydrograph at storm drain inlets
Compute hydrograph at storm drain inlets
Combine
Route through storm drain system
Combine with hydrograph from upper watershed
Compare with recorded data

① Loss rate functions are obtained from optimization of rainfall and runoff of instrumented watershed that has similar soil characteristics.
Unit hydrograph parameters are calculated as discussed in text.

② Loss rates are assumed near zero and unit hydrograph parameters are calculated as discussed in text.